1956

An analysis of the consolidation-permeability characteristics of clay soils.

Petersen, John H.

Rensselaer Polytechnic Institute

http://hdl.handle.net/10945/13905

AN ANALYSIS OF THE CONSOLIDATION-PERMEABILITY CHARACTERISTICS OF CLAY SOILS

John H. Peterson

AN ANALYSIS OF THE CONSOLIDATION - PERMEABILITY CHARACTERISTICS OF CLAY SOILS

by

John H. Petersen

A Thesis Submitted to the Faculty
of the Department of Civil Engineering
in Partial Fulfillment of the
Requirements for the Degree of
Master of Civil Engineering

Approved:

____
Adviser

Rensselaer Polytechnic Institute
Troy, New York

June, 1956

TABLE OF CONTENTS

		Page
	LIST OF TABLES	v
	LIST OF FIGURES	vi
	FOREWORD	viii
	ABSTRACT	ix
I.	INTRODUCTION	1
	A. Objective	1
	B. Historical Review	4
II.	THEORY	7
	A. Consolidation	7
	1. Consolidation Equations	7
	2. Precompression on Clays	12
	3. The Changing Value of the Coefficient of Consolidation During Consolidation	15
	4. Determination of the Coefficient of Compressibility	18
	B. Direct-Permeability Measurements During Consolidation	21
	1. Original Concepts of the Flow of Water Through Soil	21
	2. Use of the Variable-head Permeameter for Measuring Permeability in Relatively Impervious Soils	21
III.	MATERIALS AND APPARATUS	24
	A. Materials Investigated	24
	B. The Consolidation Apparatus	27
	C. Attachment for the Direct Measurement of Permeability	30
	1. Falling-head Permeameter	30
	2. Air Pressure Regulating System	35

Page

IV. EXPERIMENTAL PROCEDURES.................... 38

A. Consolidation Test................ 38

1. Preparation of Sample 38
2. Loading Procedure............. 40
3. Fitting Methods Used in Determing Extent of Primary Consolidation and Coefficient of Consolidation.............. 42

B. Testing Procedure for Direct Measurement of Permeability Using Air Pressure........................ 47

V. RESULTS AND DISCUSSION.................... 55

A. General............................ 55
B. Shape of the Resulting C_v Curves..... 70
C. Relationship between Direct and Indirect Measurements of Permeability 73

VI. CONCLUSIONS............................. 79

VII. LITERATURE CITED........................ 82

VIII. APPENDICES............................. 84

A. Test II........................... 84
B. Test III.......................... 89
C. Test IV........................... 95
D. Test V............................ 101
E. Test VI........................... 108
F. Test VII.......................... 114
G. Test VIII......................... 120

LIST OF TABLES

		Page
Table I	Information on Samples Tested	25
Table II	Maximum Air Pressures Recommended for Permeability Measurements when Using the Consolidometer as a Variable-head Permeameter	52

LIST OF FIGURES

		Page
Figure I	The Effect of Precompression in Settlement........................	9
Figure II	Pressure Conditions in a Compressible Layer that Are Changing with Time	13
Figure III	Three-Stage Consolidation Cycle.......	17
Figure IV	Fixed-Ring Type Consolidometer for a Two and One-Half Inch Diameter Sample.	28
Figure V	Consolidation Apparatus with Attached Falling-head Permeameter and Air Pressure System......................	31
Figure VI	Variable-head Permeameter Attachment for Consolidation....................	33
Figure VII	Air Pressure Regulating System for Falling-head Permeameter.............	36
Figure VIII	Air Pressure Regulating Arrangement for Permeameter......................	37
Figure IX	Typical Time Curve Using The Logarithm of Time Fitting Method from Consolidation Test T-VIII.....................	43
Figure X	Typical Time Curve Using The Square Root of Time Fitting Method from Consolidation Test T-VIII.............	46
Figure XI	Consolidation Test T-II..............	57
Figure XII	Consolidation Test T-III.............	58
Figure XIII	Consolidation-Permeability Test T-III.	59
Figure XIV	Consolidation Test T-IV..............	60
Figure XV	Consolidation-Permeability Test T-IV..	61
Figure XVI	Consolidation Test T-V...............	62
Figure XVII	Consolidation-Permeability Test T-V...	63

		Page
Figure XVIII	Consolidation Test T-VI................	64
Figure XIX	Consolidation-Permeability Test T-VI...	65
Figure XX	Consolidation Test T-VII..............	66
Figure XXI	Consolidation-Permeability Test T-VII..	67
Figure XXII	Consolidation Test T-VIII.............	68
Figure XXIII	Consolidation-Permeability Test T-VIII.	69

FORWORD

The possibility of a relationship existing between the precompression load in soil and the shape of the coefficient of consolidation curve, plotting from results of consolidation data, was first suggested by Associate Professor S. V. Best. This study was undertaken to investigate such a possibility, and to compare the direct and indirect values of the coefficient of permeability determined during consolidation.

Indebtedness is expressed to Professor E. J. Kilcawley and to Associate Professor S. V. Best for their introduction to the subject and for their interest and helpful suggestions during the course of its investigation.

Sincere thanks are further extended to Assistant Professors J. E. Munzer and William Kelleher who, throughout this study, offered valuable guidance and recommendations.

Appreciation is extended to the New York State Soil Mechanics Laboratory for providing the undisturbed samples of clay used in this study and, in particular, to Mr. Graham for his personal interest and help in obtaining the clay samples.

ABSTRACT

This investigation was conducted to study the shape of the curve and irregularities in the value of the coefficient of consolidation, C_v, determined by various methods during consolidation of undisturbed clay soil specimens, and to note whether or not a relationship exists between the digression of the curve and the amount of pre-compression in the sample. Further, to employ an air pressure variable-head permeameter for direct determinations of permeability during the laboratory consolidation process, and to apply this measured value to calculate a value of the coefficient of consolidation derived from theoretical formulas. This value of C_v determined from the directly measured permeability is compared with the other values of C_v determined by the two common fitting methods.

Seven undisturbed clay samples were consolidated. Permeability determinations were made at the end of each loading increment. The permeameter attachment was not unusual except for the air pressure regulating system, which used a line pressure of 35 p.s.i. and actual testing pressures between 0 and 15 p.s.i. A detailed study was made of the effects of various air pressures used to determine permeability, and a procedure for testing with this equipment was developed, including recommendations

for the maximum amount of air pressure to use with various consolidation loads as well as a suggested limitation for swelling of the sample and time required for swelling when pressure is applied.

The results indicate that the value of C_v determined from the two fitting methods and through use of the direct determination of permeability are similar during consolidation. The resulting curve, however, does not provide an accurate means of estimating precompression in the sample. The plotted C_v curves show a tendency to break in the vicinity of the estimated precompression for the material, but this breaking point was not distinct or conclusive in all test results. Results from the permeameter attachment prove its usefulness and adaptability. By comparing the direct determinations of permeability with those indirectly calculated, a good relationship between the two values was shown.

PART I.

INTRODUCTION

A. Objective

It is the common practice in the field of soil mechanics to determine the estimated ultimate settlement and the time-rate of settlement of a structure or fill placed over a compressible soil stratum from data based principally on the theory of consolidation. To obtain this quantitative information regarding settlement, loading, and time, undisturbed soil samples are consolidated in the laboratory. From this theoretical data and from knowledge of the estimated load which will be applied by the structure, estimates of settlement versus time may be approximated.

The settlement in a clay stratum occurs from the squeezing out of pore water under loading. The rate at which this settlement occurs is in direct relationship to the rate of displacement of pore water (the permeability of the clay material). In very fine-grained impervious soils such as clay, with low values of permeability, the process of consolidation takes place over a very long period of time. This accounts for the frequently misunderstood gradual settlement of some heavy structures placed over clay stratas.

One of the soil properties determined from the

consolidation test is the coefficient of consolidation (C_v) which, when plotted, showed in some cases a break in the curve near the load determined as the precompression load, or maximum past pressure exerted on the clay during past geologic history.

One of the objectives of this thesis is to study the shape of the coefficient of consolidation curve, plotted from results of consolidation tests on undisturbed clay samples, and to determine if a relationship exists between the breaking point of this curve and the precompression stress.

At the onset of this study, it was felt that the most variable factor influencing the coefficient of consolidation was permeability which, therefore, needed to be checked during the consolidation process. The evaluation of this factor was accomplished by measuring the permeability of the clay samples tested at the end of each loading increment, utilizing the consolidometer as a variable-head permeameter and using air pressure to provide the head necessary to produce flow in the clay sample, thereby reducing the time required for testing. By accelerating the test, the errors due to evaporation at the water surface and to temperature changes, as well as changes in void ratio, were kept to a minimum.

In general, the purpose of this thesis is to

improve upon the understanding of soil properties influencing the consolidation of undisturbed clay samples, which in turn might lead to more dependable settlement predictions.

B. Historical Review

The first marked advancement in the study of the gradual adjustment occurring in weighted soils appeared as the Theory of Consolidation, developed and published by Dr. Karl Terzaghi in 1925.[14] To date, a great deal has been published regarding that theory and its relationship to settlement analysis, and no attempt will be made here to repeat what can be found in many soil mechanics texts.[1, 3, 8, 13, 15]

To apply the consolidation theory to actual laboratory tests, an apparatus called an oedometer was designed by Terzaghi.[12] Later, Dr. Glennon Gilboy designed a consolidometer[6] similar in many respects to the consolidation equipment used today and similar, except in diameter of sample tested, to the consolidometers used in this study.

One important aspect of the familiar logarithm of pressure-void ratio consolidation curve is the determination of the degree of precompression. The method of determining the maximum intergranular pressure to which an undisturbed soil sample had been subjected, and the significance of this information, was authored in 1936 by Dr. Arthur Casagrande.[2] For soils which had been precompressed by such forces as the load from glaciers, upheaval and subsequent erosion, or by changes in capillary pres-

sures, settlements from loads below the range of precompression were much smaller than those occurring in soils which had not been subjected to past loading.

After reviewing much of the history and investigations conducted in the past on consolidation, the author was unable to find any studies which made an attempt to relate the degree of precompression in undisturbed clay samples to the shape of the coefficient of consolidation curve. Therefore, any history covering this particular phase of the present investigation is incomplete.

In connection with the use of the consolidometer for measuring permeability during a laboratory consolidation test, Gilboy reported in 1933 that, on two samples selected at random, close agreement between direct and indirect determination was found.[7] More recently, Professor Donald W. Taylor conducted a great number of comparisons between direct and indirect permeability determinations on remolded Boston Blue Clay which are worthy of note.[13] The direct permeability measurements showed a considerable scattering, using various load increment ratios, but no general trend as a result of that variation. By calculating the indirect value of permeability from the consolidation data, Taylor found that a reasonable agreement exists between the direct and indirect determinations. In this study, it is proposed to carry the check one step further

and to utilize the directly measured permeability to determine values of the coefficient of consolidation, thereby obtaining another plot of the changing coefficient of consolidation curve versus pressure.

The fitting methods used in this study to determine the coefficient of consolidation from the laboratory time curves were the two generally accepted methods; namely, the "Logarithm of Time Fitting Method" developed by Casagrande and the "Square Root of Time Fitting Method" devised by Taylor.[12] Results from both types of fitting methods were combined in some tests to obtain a more representative e - log p relationship, particularly in those tests which, under low load increments, did not result in the characteristic curve necessary to the use of the logarithm of time fitting method.

PART II.

THEORY

A. Consolidation

1. Consolidation Equations

In the design of a foundation, the limitations of settlement are often the controlling factors. In general, settlement is caused by two conditions. One is the result of lateral deformation without change in volume, and the other is a gradual volume change due to the drainage of pore water or, in other words, consolidation. The lateral deformation is minor and is usually not considered;[3] consequently, consolidation is of primary concern and is used as the basis for most settlement predictions.

The theory of consolidation is based on one-dimensional compression which is confined against movement in any lateral direction and is taking place in completely saturated soil. This is essentially true inasmuch as clay stratas will not deform laterally and are completely saturated below the ground water level. Other assumptions used to simplify the solution are that the soil particles and water are incompressible, that the flow of water follows Darcy's Law, and that the soil mass is homogeneous. The one-dimensional theory also considers that the movement of pore water takes place in the vertical direction only, moving either upward or downward, depending on the boundary

conditions. This assumption is reasonably accurate for homogeneous stratas bounded by pervious layers, but considerable variation will exist in the time settlement ratio if the strata is heavily varved, permitting horizontal drainage.

The pressure distribution in a compressible layer between two permeable layers at any instant of time after loading is represented graphically by Figure I. Before the loading is applied, the effective pressure p_e, is equal to p_w, the pressure taken by the water. At any time after the instant of loading, compression of the soil particles occurs, p_w becomes less than the effective pressure, and a portion of the load is carried by the water while part is transferred to the soil skeleton. As shown by Figure I, this varies with the depth z, which indicates that a greater part of the load p_w, termed hydrodynamic excess, is carried by the water nearer to the mid-section of the strata. As time elapses, more of the load is carried by intergranular pressure until equilibrium is reached, the hydrodynamic excess is zero, and the load is carried entirely by the soil skeleton.

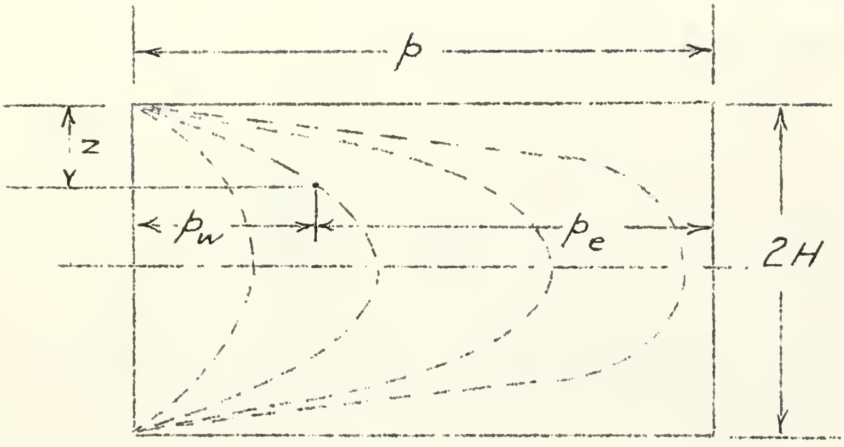


FIGURE I

Pressure Conditions in a Compressible Layer
which are Changing with Time

Since the derivation of the basic equation from the concepts outlined above is already published in many texts and articles on the subject, 4, 8, 12, 14 it is not included here. This fundamental differential equation is written as follows:

$$\frac{k}{\gamma_w} \frac{\partial^2 p_w}{\partial z^2} = \frac{a_v}{1+e_a} \frac{\partial p_w}{\partial t}$$

By combining terms, this equation becomes

$$\left[\frac{k(1+e_a)}{a_v \gamma_w} \right] \frac{\partial^2 p_w}{\partial z^2} = \frac{\partial p_w}{\partial t}$$

Since the terms in the brackets are designated the coefficient of consolidation, C_v, the equation may be expressed as:

$$C_v \frac{\partial^2 p_w}{\partial z^2} = \frac{\partial p_w}{\partial t}$$

The coefficient of consolidation is equal to

$$C_v = \frac{k(1+e_a)}{a_v \gamma_w}$$

Where k is the coefficent of permeability, e_a is the average void ratio at 50 per cent of primary consolidation, γ_w is the unit weight of water, and a_v is the coefficient of compressibility equal to

$$a_v = -\frac{de}{dp} = \frac{e_1 - e_2}{p_2 - p_1}$$

The solution to the fundamental differential equation, using the boundary conditions for one-dimensional flow, was through use of the Fourier Series. The fundamental concepts underlying the theory, as well as a solution to the equation under conditions of loading and drainage met in the field, are compiled in "Notes on Soil Consolidation" by Professor E. J. Kilcawley.[8] The relative value at any point in the sample is expressed in

terms of the percentage of consolidation, U_z, by the equation:

$$U_z = 1 - \sum_{m=0}^{m=\infty} \frac{2}{M} \sin \frac{Mz}{H} \epsilon^{-M^2 T}$$

where T is a dimensionless number called a time factor and is equal to

$$T = \frac{C_v t}{H}$$

and C_v is the coefficient of consolidation, t is the time interval, and H is one-half the thickness of the strata or sample being tested.

The average consolidation, U, which has occurred throughout the stratum as a whole and is of greater practical significance than the values at various depths, is expressed by the following equation:

$$U = 1 - \sum_{m=0}^{m=\infty} \frac{2}{M^2} \epsilon^{-M^2 T}$$

The equation for U, above, has been solved providing a curve for determining the time factor, T, for any percentage of consolidation, U.[12] This eliminates the need for long mathematical calculations in determining T and is used in this investigation.

As mentioned in Part I, two fitting methods are employed in this study to determine the limits of the primary compression corresponding to the theory and for determining the coefficient of consolidation. The two equations used with these fitting methods are as follows:

Using the Logarithm of Time Fitting Method:

$$C_v = \frac{0.197\,(H)^2}{t_{50}}$$

Using the Square Root of Time Fitting Method:

$$C_v = \frac{0.848\,(H)^2}{t_{90}}$$

The .197 and .848 are the values of T for fifty and ninety per cent of consolidation, respectively. The t_{50} and t_{90} are the number of seconds required to reach the points of fifty and ninety per cent consolidation, and H is one-half the thickness of the sample in centimeters. The coefficient of consolidation, C_v, is then expressed in centimeters squared per second.

2. <u>Precompression in Clays</u>

The effect of precompression on settlement is best shown by drawing an example of the relationship between the changing void ratio and an increasing load represented as the logarithm of pressure on a clay stratum.

Figure II represents such a relationship. Consider the clay layer as having been precompressed at one time in its natural history by a pressure equal to p_3, shown as point A on the curve, and later reduced by erosion to the present overburden pressure p_1, shown as point B. Consider next that the pressure on the strata is again increased by a building load to p_2, represented by point C on the curve. The compression under the building load will occur along what is termed the recompression portion of the curve from points B to C and is equal to Δ_1. If the clay had not been precompressed and was consolidated only under the present overburden pressure p_1, represented by point D on the virgin compression curve, the additional stress caused by the building would create a compression from points D to E equal to Δ_2. It is evident that settlement in the precompressed clay would be much less.

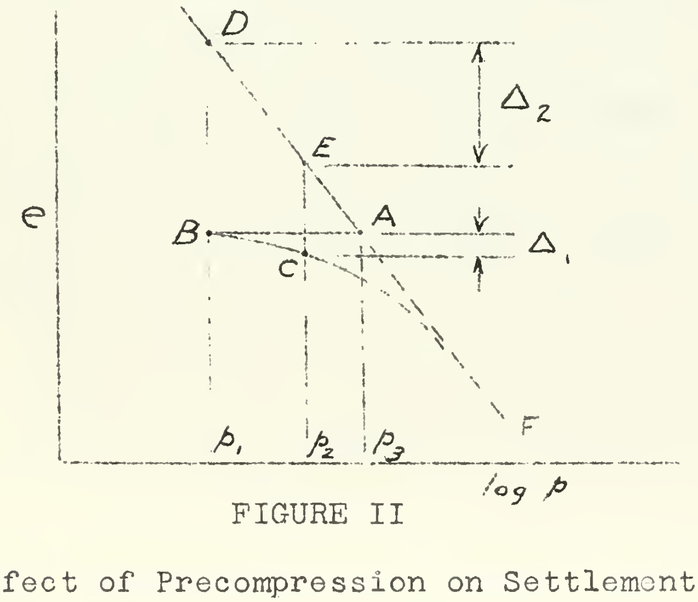


FIGURE II

Effect of Precompression on Settlement

In testing undisturbed clay samples taken from below the ground surface, the consolidation stress-strain (e - log p) curve will follow the characteristic shape. This will be an initial recompression resulting in small changes in void ratio, then a bend downward to follow the relatively straight line of the virgin compression curve. The typical curve is shown as line BCF on Figure II.

From the characteristic curve, the magnitude of precompression load is determined. To locate this point, Casagrande devised a graphical method which provides a rough estimate of this value, the accuracy of which depends on the care taken in sampling and testing.[2] This method involves locating the point of maximum curvature on the e - log p curve by eye, then drawing a horizontal line and a tangent line to the curve through this point. Another line is drawn bisecting the angle between the horizontal line and the tangent line, while yet another line is drawn upward following the straight line portion of the virgin curve. The intersection of this line with the line bisecting the angle locates roughly the magnitude of precompression for the sample tested.

This method, based on test data, is used in this study for estimating the precompression on the undisturbed

samples tested. It is also used to determine whether or not a relationship exists between the precompression load in the samples and the changing coefficient of consolidation during consolidation of the samples.

3. <u>The Changing Value of the Coefficient of Consolidation during Consolidation</u>

The coefficient of consolidation, C_v, is defined as the ratio between the permeability and compressibility of the soil.[14] Its change with respect to the loading increments employed in testing varies with these two factors. In general the values of k, permeability, and a_v, coefficient of compressibility, decrease gradually with increasing values of pressure, but at different rates. The change in the value of C_v, occurring at each loading increment is small but it does show certain characteristic trends.

During the initial loading increments of an undisturbed precompressed clay the compressibility factor, a_v, will decrease rather rapidly at first and then level off during the recompression period. As the loading increases beyond the point of precompression, the factor a_v, decreases more rapidly for each load increment. The change in permeability will be only slight during the reloading portion of the consolidation stress-strain curve but for increments of load after precompression the rate of change

in k will increase and follow a straight-line relationship similar to that of the decreasing void ratio plotted against the logarithm of pressure. The greatest changes between the values of a_v and k, between loading increments, will occur during the transition period from the recompression part of the curve to the virgin compression curve, resulting in a change in the C_v curve when plotted against the logarithm of pressure. Although not an accurate means of determining the magnitude of precompression, the changing values of C_v with pressure may provide a rough check of the other common methods employed in determining precompression.

To show the changing effect of these two factors, consider a void ratio-logarithm of pressure relationship from a typical consolidation test on clay which has been precompressed at one time, as shown in Figure III. The original in situ condition is indicated by point O, which is the pressure of the overburden. Point O' represents a higher stress under which the clay had been fully consolidated by precompression. During the process of sampling and transferring to the consolidometer, the void ratio will remain practically the same; however, the effective vertical pressure is considerably reduced. This process is shown in Figure III as the first stage, sampling. When the sample is subjected to loading in the laboratory,

the void ratio will gradually reduce to a point between a and b, depending on the remolding and damage done to the sample during its removal from the ground. This reloading is labeled the second stage in Figure III and is referred to as recompression. Beyond the second stage, the void ratio is reducing at a faster rate per loading increment, constituting the third stage of the cycle, and results in a faster rate of change in permeability per loading increment.

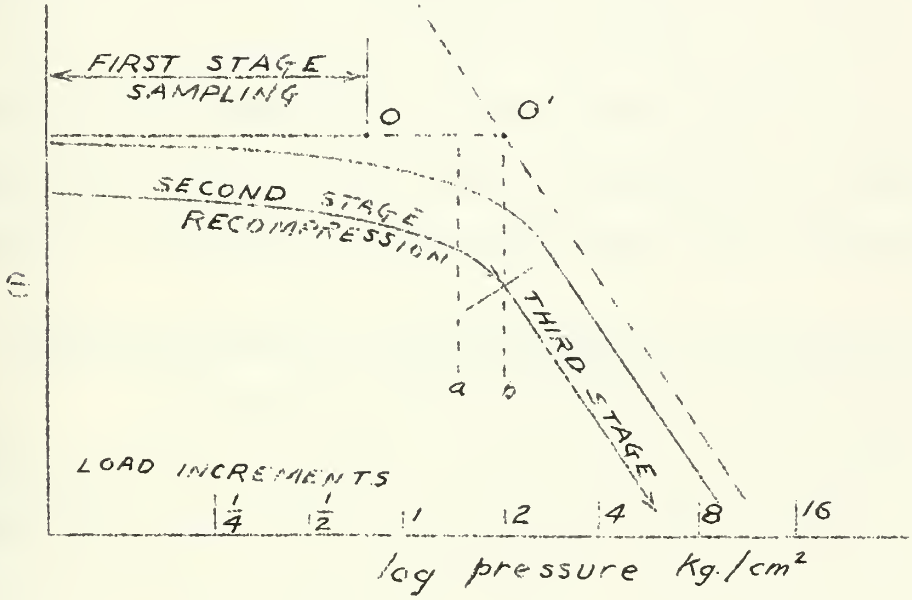


FIGURE III

Three-Stage Consolidation Cycle

As mentioned in Section A of this Part, C_v is found through the use of two methods: The Logarithm of Time and The Square Root of Time Fitting Methods. To

further check the shape of the C_v curve with respect to pressure, the direct measurements of permeability were obtained during consolidation and used in the following equation to calculate C_v:

$$C_v = \frac{k_d (1 + e_k)}{a_v \, \gamma_w}$$

In this equation, k_d is the directly measured value of permeability obtained by transforming the consolidometer into a falling-head permeameter and using air pressure to provide the necessary head. e_k is the void ratio of the sample determined at the time of the permeability test, and a_v is the coefficient of compressibility determined for the particular load increment.

4. <u>Determination of the Coefficients of Compressibility</u>

To show the characteristics of the test results more clearly, the void ratio-logarithm of pressure plot is used throughout the investigation. The straight-line portion of the compression curve is represented by the equation

$$e_2 = e_1 - C_c \log \frac{p_2}{p_1} \qquad (1)$$

where e_1 is the void ratio corresponding to pressure, p_1, and e_2 is the void ratio at p_2. The slope

of the straight line on the semi-logarithmic plot is an emperical coefficient called the compression index, C_c. This same equation may be written as follows:

$$e_1 - e_2 = C_c\,(\log p_2 - \log p_1) \qquad (2)$$

The value of C_c is constant on the straight-line portion of the e - log p curve and is not applied to the curved portion of the curve for lower loading increments. However, by drawing a tangent to the curve through the average pressure point for the lower load increments, a compression index, C_c', may be obtained as the slope of this tangent line. The average pressure point is equal to

$$p = \frac{1}{2}(p_1 + p_2)$$

Since Terzaghi's theory is based on compression resulting from primary consolidation, and the e - log p relationship is developed from the 100 per cent primary consolidation on the laboratory compression-time curves, the slope of a tangent line drawn through this point is representative of the compression index for the primary consolidation.

With the value of the coefficient of compressibility given by the equation

$$a_v = \frac{e_1 - e_2}{p_2 - p_1} \tag{3}$$

the value of a_v for each load increment may be determined for almost all loadings by combining equation (2) with equation (3), and by changing C_c to $C_c{}'$ which will vary for the lower loading increments. The equation then becomes

$$a_v' = \frac{C_c'(\log p_2 - \log p_1)}{p_2 - p_1} \tag{4}$$

p_1 and p_2 are the pressure increments in grams, and $C_c{}'$ is the revised compression index for the same pressure increment.

Since it is impossible to draw the curve representing the entire initial load increment from 0 to 250 grams per square centimeter on the semi-logarithmic plot, the determination of $a_v{}'$ for this loading increment is not accurate by equation (4) and the equation is therefore used only for loading increments above 250 grams per square centimeter.

B. Direct Permeability Measurements during Consolidation

1. Original Concepts of the Flow of Water through Soil

The coefficient of permeability is a soil property indicating the ease with which water flows through soil pores. The importance of this soil property is well known and, as previously discussed, is one of the primary factors influencing consolidation and the time rate of settlement of structures over clay stratas.

The law concerning the flow of water through soils was developed by Darcy and is shown by the following equation:

$$Q = kiA$$

where Q is the rate of discharge, A is the cross-sectional area of the soil mass where the discharge occurs, k is the coefficient of permeability, and i is the hydraulic gradient.[5]

Professor T. William Lambe, in an article regarding the permeability in fine-grained soils, found that the major factors influencing the permeability are (1) soil composition, (2) permanent characteristics, (3) void ratio, (4) structure, and (5) the degree of saturation.[9]

2. Use of the Variable Head Permeameter for Measuring Permeability in Relatively Impervious Soils

Since the flow of water through impervious soil,

such as clay, is very small, an accurate measurement of the water passing through a sample is difficult to obtain. To alleviate this difficulty in measuring the permeability of a clay sample under consolidation, the consolidometer is transformed into an upward flow variable-head permeameter, as described in Part III, Section C. The method employs the use of a standpipe to provide the necessary head of water for percolation. The quantity of water is measured indirectly by observing the rate of fall in the water level in the standpipe.

Taylor has shown that the rate of flow may be expressed as the area of the standpipe, a, multiplied by the velocity of the fall.[12] Since the velocity of fall is equal to $-\frac{dh}{dt}$ at any time, Darcy's Law may be expressed as

$$-a\frac{dh}{dt} = k\,i\,A$$

Substituting $\frac{h}{L}$ for the gradient, where h is the pressure head and L is the length of drainage path, it becomes

$$-a\frac{dh}{dt} = k\frac{h}{L}A$$

By considering h_o as the head measured at zero time, and h_1 as the head measured after a time interval,

t, and then integrating the equation, it becomes:

$$-a\int_{h_0}^{h_1}\frac{dh}{h} = k\,\frac{A}{L}\int_0^t dt$$

$$-a\,\ln\frac{h_0}{h_1} = k\,\frac{A}{L}\,t$$

and

$$k = \frac{aL}{At}\ln\frac{h_0}{h_1}$$

or

$$k = 2.3\,\frac{aL}{At}\log_{10}\frac{h_0}{h_1}$$

By using air pressure in the permeameter standpipe to increase the head, the equation for permeability may be written as follows:

$$k = 2.3\,\frac{aL}{At}\log_{10}\frac{h_0 + h_p - h_c}{h_1 + h_p - h_c}$$

in which the h_p is the average air pressure applied during the test in terms of centimeters of water, and h_c is the height of capillary rise. This equation was used in the direct permeability determinations made throughout this investigation.

PART III

MATERIALS AND APPARATUS

A. Materials Investigated

The soil samples used in this investigation were arbitrarily chosen undisturbed clay soil specimens furnished by the New York State Soil Mechanics Laboratory. The undisturbed samples were removed from the ground with a thin-wall drive sampler of three and one-half inches diameter, sealed with caps and wax to prevent the loss of moisture. Table I gives a tabulation of the soils tested, together with data on the location of the test borings. More detailed information on the characteristics of each test is presented in the appendices.

Infrared analysis conducted by D. M. Welton during an investigation of "Infrared Analysis Applied to Clay Minerals", June 1956, found that the Saint Lawrence samples (Test numbers T-II, T-III, and T-IV) indicated a predominance of illite with possibly some hectorite and also a distinct quantity of quartz impurities. The same analysis conducted on the Phoenicia-Stony Cove sample (Test number T-V) indicated a high percentage of montmorillonite clay and some quartz impurities. An infrared analysis made on the Albany clay sample (Test numbers T-VI and T-VII) and the Massena sample (Test number T-VIII) most closely indicated an illite clay mineral with both curves

TABLE I

Information on Samples Tested

Test No.	General Location	No. of Test Boring	Depth in Feet	Visual Identification
T-II	St. Lawrence Power Development, Richards Landing Dike	S-RLUH-4	53	Medium Gray Clay and Silt with Trace of Sand and Gravel
T-III	St. Lawrence Power Development, Long Sault Dike #3	S-LSUH-2	14	Medium Gray Clay and Silt
T-IV	St. Lawrence Power Development, Long Sault Dike #3	S-LSUH-2	37	Medium Gray Clay and Silt
T-V	Phoenicia-Stony Cove Bridge Site	1-146-15 T-8	39	Medium Reddish-Brown Clay and Silt with Trace of Fine Sand
T-VI	Albany-State Education Bldg.	1-151 T 1-17	24 to 25.5	Hard Blue Clay with very thin Silt Varves
T-VII	Albany-State Education Bldg.	1-151 T 1-17	24 to 25.5	Hard Blue Clay with very thin Silt Varves
T-VIII	Massena-South Side Arterial	Station 38-75 H-21	10.6 to 10.9	Soft Clay and Silt with Trace of Sand and Gravel

being very much alike. The Albany sample did show a little more quartz, however, which would account for a higher silt content in that sample.

A differential thermal analysis conducted by R. A. Litke during an investigation of "Clay Minerals, Organic Ions, and Differential Thermal Analysis", June 1956, classified the Albany sample (Test numbers T-VI and T-VII) as most closely resembling a chloretic mica type of clay mineral.

B. <u>The Consolidation Apparatus</u>

In this investigation, a medium capacity consolidation apparatus, manufactured by Soiltest Incorporated, was used. The apparatus is capable of loading two consolidometers simultaneously. Each consolidometer is loaded by a double-lever system which is adjustably counterbalanced to compensate for the dead-load weight of the levers and loading system. Fulcrum points in the system are knife edges which are machined and hardened. The frame is made up of welded structural steel sections, and the loading beam on which the consolidometers rest consists of two six-inch "I" beams welded onto the frame. The entire assembly was leveled by adjusting four corner screws. A two-inch square loading bar is connected between the upper lever system for counterbalancing and the lower lever system for loading by two seven-eighths-inch steel bolts. A hardened steel semi-sphere and four springs mounted on the loading bar hold the loading disc in place over the upper porous stone, insuring concentric loading at all times during the test.

The consolidometers used were of the fixed-ring type shown in Figure IV, also manufactured by the Soiltest Company. The sample rings for the consolidometers were two and one-half inches in diameter. The consolidometer base was machined out to provide room for a one-half-inch-thick porous stone, together with grooves below for

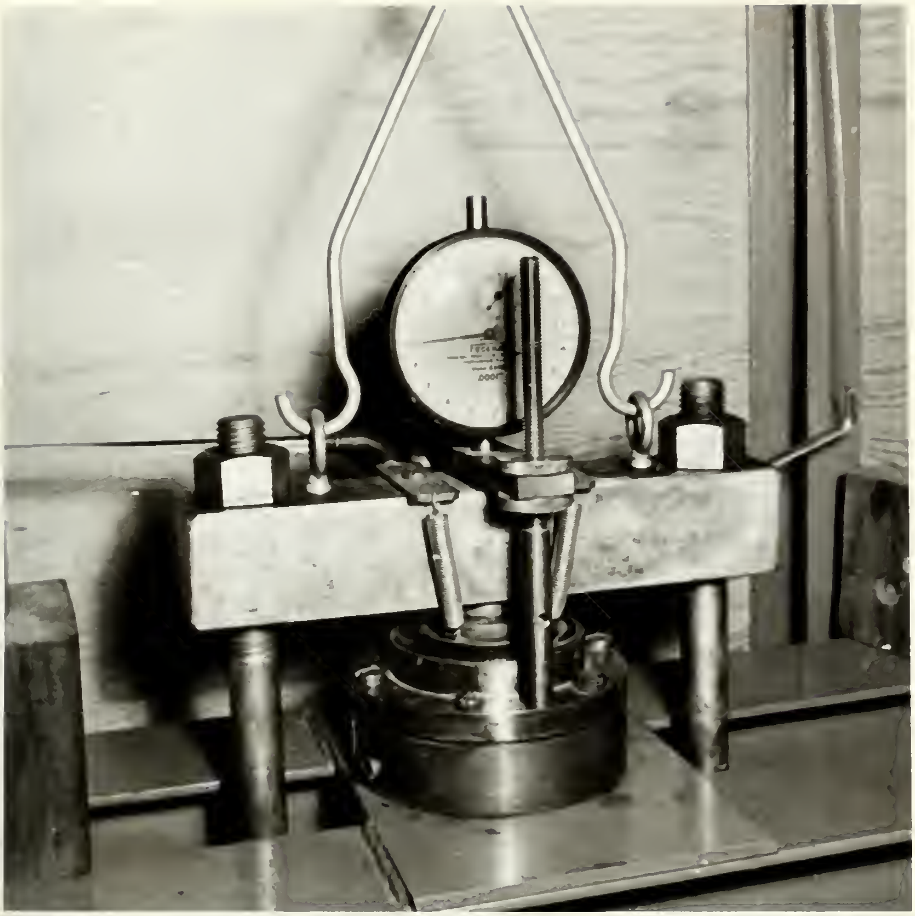


FIGURE IV

Fixed-Ring Type Consolidometer for

Two and One-half Inch Diameter Sample

drainage. These grooves lead into two outside screw connections, providing the means for measuring permeability and for drainage. Another porous stone is placed above the sample for upward drainage. The outside diameters of the porous stones are slightly smaller than the inside diameter of the sample ring, providing an adequate bearing area and sufficient clearance to prevent side friction.

Also included in each assembly is a gutter ring which is placed over the sample ring and gasket to hold the sample rigidly in place and to prevent leakage from below the sample between the base and gutter ring of the consolidometer. The gutter ring is secured to the base by six bolts and has a diameter larger than that of the loading disc so that water may stand within the gutter to insure saturation of the sample and prevent the loss of moisture by evaporation.

To measure the reduction in thickness of the sample during consolidation, as well as its thickness at the time of direct measurements of permeability, a dial micrometer with 0.0001-inch graduations is supported above the loading bar, as shown in Figure IV. The measuring point rests directly over the sample on the loading bar.

C. Attachment for the Direct Measurement of Permeability

1. Falling-head Permeameter

In order to measure the coefficient of permeability directly during the consolidation of a clay sample, it was necessary to transform the consolidation apparatus into a falling-head permeameter, using an upward flow of water through the sample. Since the clay samples tested were very impervious, it was desirable to avoid using either very high water columns or long time intervals in producing sufficient flow through the sample. Consequently, an attachment utilizing air pressure was constructed and was attached to the consolidation apparatus frame, as shown in Figure V.

The main features of this permeability attachment were modeled after a similar device developed by Rutledge, pictured with his article on testing equipment which was published in 1935.[8] The attachment consists of a standpipe water column to measure the drop in head from the beginning to the end of the test, a water reservoir to refill the standpipe between tests, a mercury manometer with adjustable scale to measure the air pressure, and an air pressure supply and regulating system to provide a constant source of air pressure. The air pressure on the supply line was approximately thirty-five p.s.i., and the pressure used in the permeability attachment varied from one to fifteen p.s.i., depending on the vertical downward pressure of the load increment.

FIGURE V

Consolidation Apparatus with Attached

Falling-head Permeameter and Air Pressure System

A detailed drawing of the permeability attachment is given in Figure VI. A plywood panel of one-half inch thickness, eighteen inches width, and fifty-two inches length was used to mount the standpipe for measuring the drop in head, the mercury manometer for measuring air pressure, and the water reservoir and supply lines. Inasmuch as a very small volumn of water will flow through the sample during a permeability test, and a large drop in head is desirable for calculating permeability by use of the variable-head method, capillary tubing with an inside diameter of 1.6 millimeters was used for the standpipe. This provided a substantial drop in head over a relatively short period of time and, consequently, reduced errors from evaporation, temperature changes, and compression of the sample during the testing period. Directly below the standpipe is located a two-way stopcock with capillary tube openings. One opening connects upward to the standpipe and two lead downward to the consolidometer base and to the water supply reservoir, respectively. By turning the stopcock one way, water is admitted from the supply reservoir, and by turning the stopcock in the opposite direction, flow is permitted from the standpipe into the base of the consolidometer and sample. To eliminate the possibility of swelling in a rubber hose connection between the two-way stopcock and

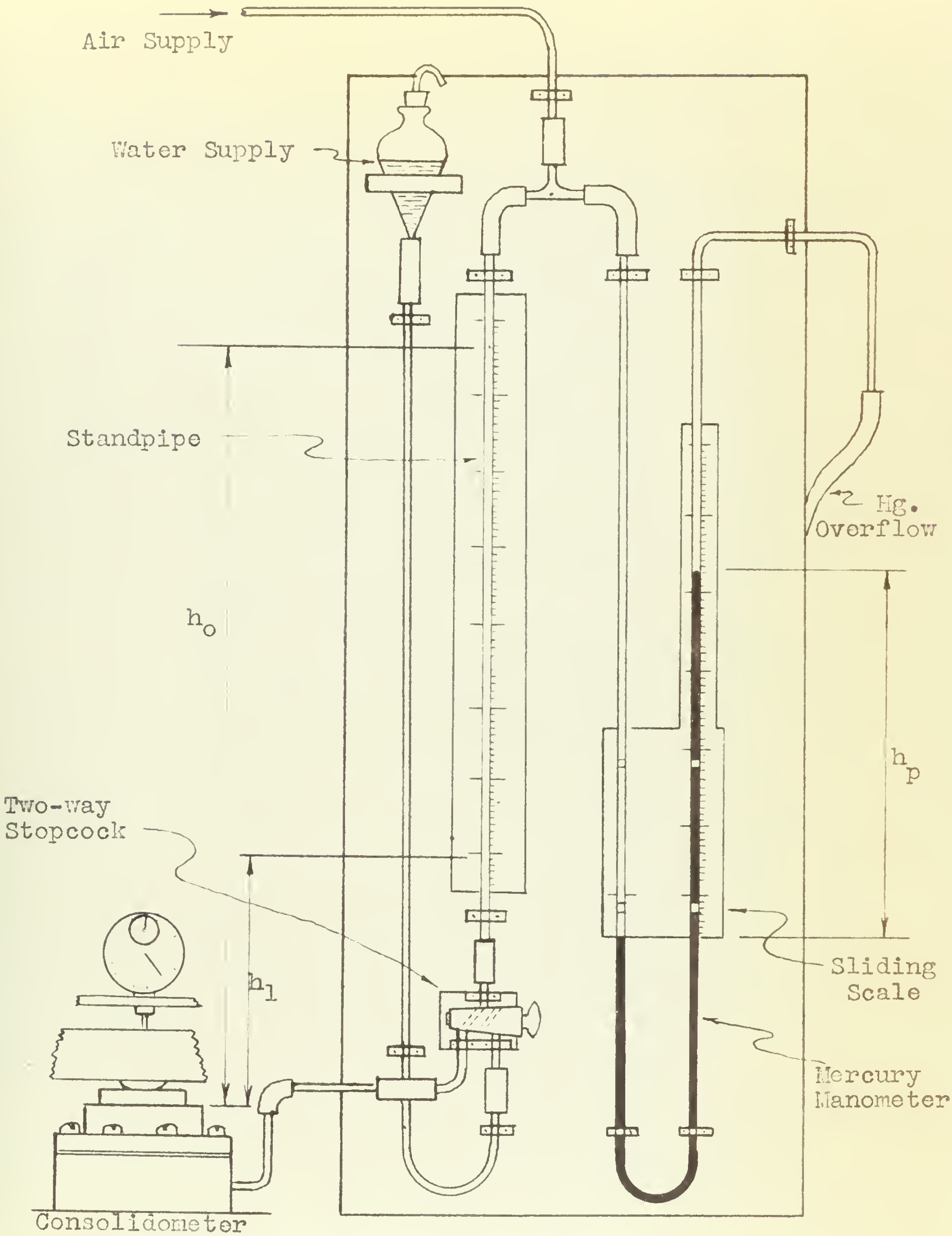


FIGURE VI

Variable-head Permeameter Attachment for Consolidometer

the consolidometers, capillary glass tubing was substituted for this tie-in, limiting the use of rubber hose to short connecting sections.

From the one-fourth inch copper tubing supplying the air pressure at the top of the permeability apparatus, a "T" connection was installed to direct the air pressure to the top of the water column and to the mercury manometer for pressure readings. An overflow tube and container to catch any mercury which might escape from the top of the manometer during a sudden change in air pressure was installed and is considered advisable. Capillary tubing was used for the mercury manometer.

A centimeter scale was used to measure the drop in head in the standpipe from h_o to h_1, or height of water column at zero time and height at the end of the test time, t. Both heights were measured from the top of the free water surface above the sample, as shown in Figure VI, which was the top of the consolidometer gutter ring. The scale on the mercury manometer was in centimeters, and was later converted to its equivalent head in centimeters of water, and added to the readings for h_o and h_1.

Care was used throughout the system to obtain water-tightness and air-tightness. All pipe connections under air pressure were lead-sealed joints. Other rubber hose connections between tubing were cemented and then

wrapped in rubber strips to curtail leaks at these weak points.

2. Air Pressure Regulating System

An air pressure regulating system was used in conjunction with the falling-head permeameter attachment, as shown in Figure VII. Air pressure was reduced from the laboratory supply line by a Harris Line Regulator to obtain the low pressures used in this investigation. With the regulator, air pressure on the system could be increased from zero, when the regulator valve was open, to any desired pressure on the standpipe column of water. A thirty p.s.i. pressure gauge was installed in the outlet line from the pressure regulator, giving a rough indication of the line pressure in the permeability system. A shut-off valve and pressure release valve were installed immediately behind the pressure gauge, so that the pressure in the system could be released to refill the standpipe without effecting the pressure regulating valve setting. A diagram of the air pressure regulating system is shown in Figure VIII.

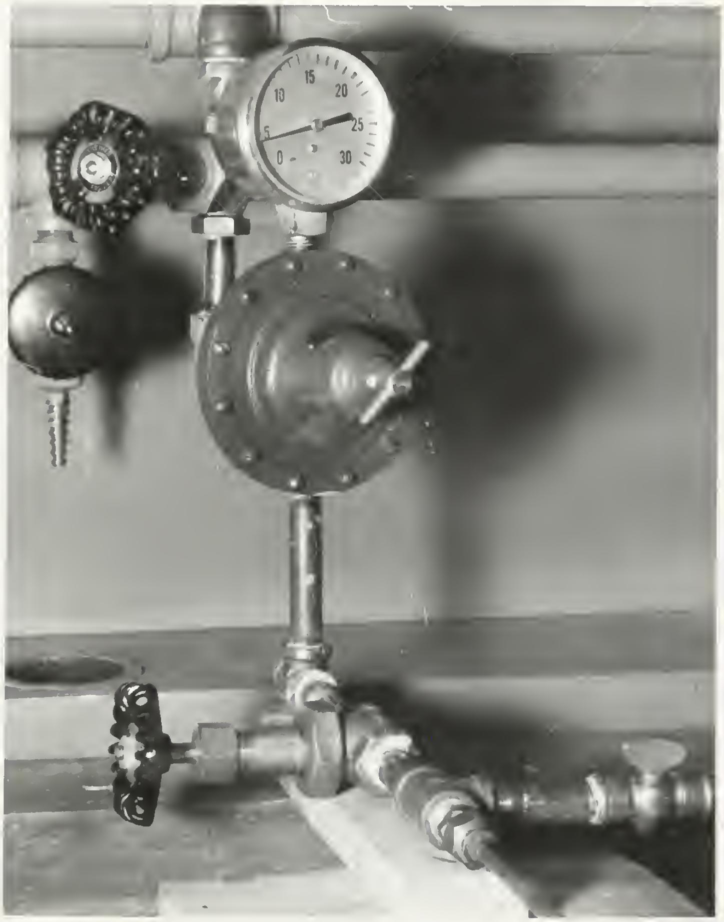


FIGURE VII

Air Pressure Regulating System for Falling-head Permeameter

A - Compressed Air Supply

B - Main Shut-off Valve

C - No. 40-A Harris Line Regulator

D - Air Pressure - 30 p.s.i. Gauge

E - Air Pressure Cut-out Valve for Refilling Standpipe

F - Air Pressure Release Valve

G - $\frac{1}{4}$ inch Copper Tubing Air Supply Line to Permeameter

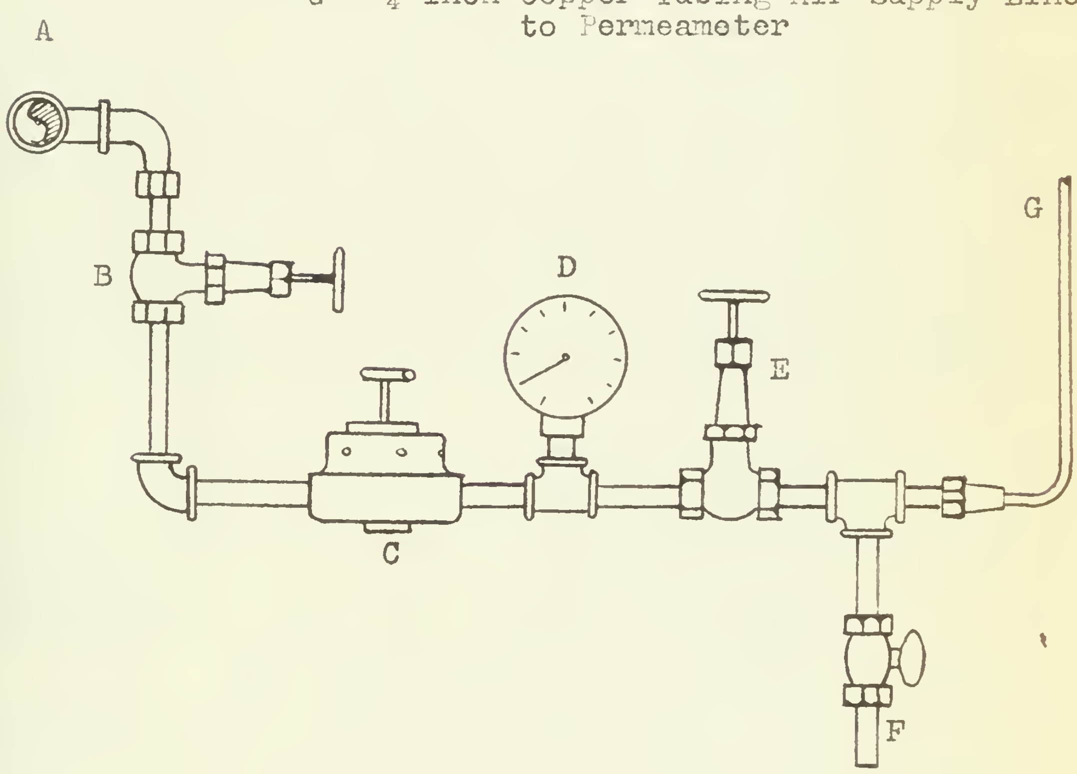


FIGURE VIII

Air Pressure Regulating Arrangement for Permeameter

PART IV

EXPERIMENTAL PROCEDURES

A. Consolidation Test

1. Preparation of Sample

In view of the permeability tests which were run on the samples during consolidation, the preparation of a sample for loading was altered slightly from the procedure outlined in the R.P.I. Soil Mechanics Laboratory Manual.[11] This change made it possible to obtain complete saturation in the lower drainage passages of the consolidometer and standpipe, thereby preventing entrapped air from effecting the permeability determinations. The following procedure was followed in preparing the samples for consolidation:

(1) A short copper tubing standpipe was installed in one of the drainage openings in the base of the consolidometer. The other opening was plugged.

(2) The consolidometer was assembled first by placing in the base unit one porous stone, which had previously been soaked in distilled water, and raising the water level above the stone. Next, the sample ring, gasket, and gutter ring were placed over the base of the consolidometer and bolted down securely. Tightness of the bolts was important, since any water leakage from the lower chamber during a pressure permeability test would

have produced erroneous results.

(3) The assembly was filled with distilled water to the top of the sample ring by running water through the standpipe, thereby removing any air which might be in the system. By bringing the water to this level before lowering the sample into place, air was prevented from being trapped below the sample. When the sample was lowered, the water was forced out through the drainage standpipe. A piece of filter paper was cut to size and placed over the lower porous stone, again taking care to avoid trapped air bubbles.

(4) With the consolidometer now ready for the sample, the sealed soil sample tube was opened and placed into the frame used for removing undisturbed specimens. The soil was forced out of the tube, by a hydraulic jack, to a length of about one and one-half inches. The sample from the tube was carefully cut off and placed on a flat surface for sizing.

(5) A sample sizer was used to cut the sample to the exact diameter of the consolidometer sample ring. The trimmings from this were used for determining the initial moisture content. The two ends were cut flush with a wire saw.

(6) Next, a sample sizer with enclosed soil was carefully centered over the thickness ring, which measured exactly 0.585 inches in depth. Using a brass

plunger, the sample was forced into the thickness ring so that a small amount extended from both ends. The top and bottom of the sample were then trimmed to the exact thickness of the ring, ascertaining that the surfaces were plane.

(7) Two glass plates were placed on either side of the sample and ring to prevent loss of moisture, and then the weight of the sample was determined.

(8) The thickness ring with sample was centered over the sample ring in the consolidometer, which was topped with distilled water. Again, by using the plunger, the sample was slowly lowered into the consolidometer sample ring, allowing the excess water to drain out through the standpipe.

(9) The top porous stone and filter paper, both of which had been soaked in distilled water, were placed over the sample. The entire assembly was then immediately centered under the loading bar and disc. Distilled water was added to the area between the upper porous stone and gutter, assuring complete saturation of the sample.

2. Loading Procedure

After the consolidometer with sample was centered directly below the loading disc, the following procedure for loading was followed:

(1) The dial micrometer was set in place on the dial holder supported in the consolidometer and adjusted

to zero reading at the beginning of its release run.

(2) The first loading increment of $\frac{1}{4}$ Kg. per cm.^{2} was applied to the sample by adding a weight to the lower lever arm. Subsequent loads applied were $\frac{1}{2}$, 1, 2, 4, 8, and 16 Kg. per cm.,^{2} in that order. Some samples tested were only carried to 8 Kg. per cm.^{2} Compression dial readings were taken at total elapsed times of $\frac{1}{4}$, $\frac{1}{2}$, 1, 2, 3, 5, 10, 15, 30, 60, 120, 240, and 1440 minutes, and were recorded so that consolidation curves could be drawn. In some cases, final recording time varied somewhat; however, sufficient readings were taken throughout the test so that a representative consolidation curve was plotted. After each loading increment had been applied for a period of about 24 hours, permeability tests were conducted on the sample. Details of the permeability test are discussed in Part IV, Section B. A series of these additional tests required four hours. Consequently, the next loading increment was applied after a period of about 48 hours.

(3) At the end of this period, the next loading increment was applied and the same general procedure was followed. In view of the additional time required for permeability tests, the entire consolidation-permeability test lasted in the neighborhood of two weeks.

(4) At the end of the final loading increment, after all dial readings and time intervals had been record-

ed, the load was decreased to 2 Kg. per cm.^{2} and then to $\frac{1}{4}$ Kg. per cm.,^{2} allowing several hours for each rebound load.

(5) Next, the sample was removed from the consolidometer, placed in a dish and weighed. The sample and dish were then moved to a drying oven and were allowed to dry for a period of 48 hours, after which the sample and dish were again weighed to determine the dry weight of the specimen.

(6) During the consolidation process of the sample, the true specific gravity of the material was determined from sample trimmings left over from the sizing operation.

3. Fitting Methods used in Determining the Extent of Primary Consolidation and the Coefficient of Consolidation.

As previously mentioned, two well-known fitting methods were employed in this investigation to determine the coefficient of consolidation value for each loading increment. The time-compression relationship of each loading increment was plotted for both the Logarithm of Time and the Square Root of Time Fitting Methods.

A typical laboratory time curve, using the Logarithm of Time Fitting Method, is shown in Figure IX. This curve is similar in shape to the theoretical consolidation curve which uses the logarithm scale for plotting

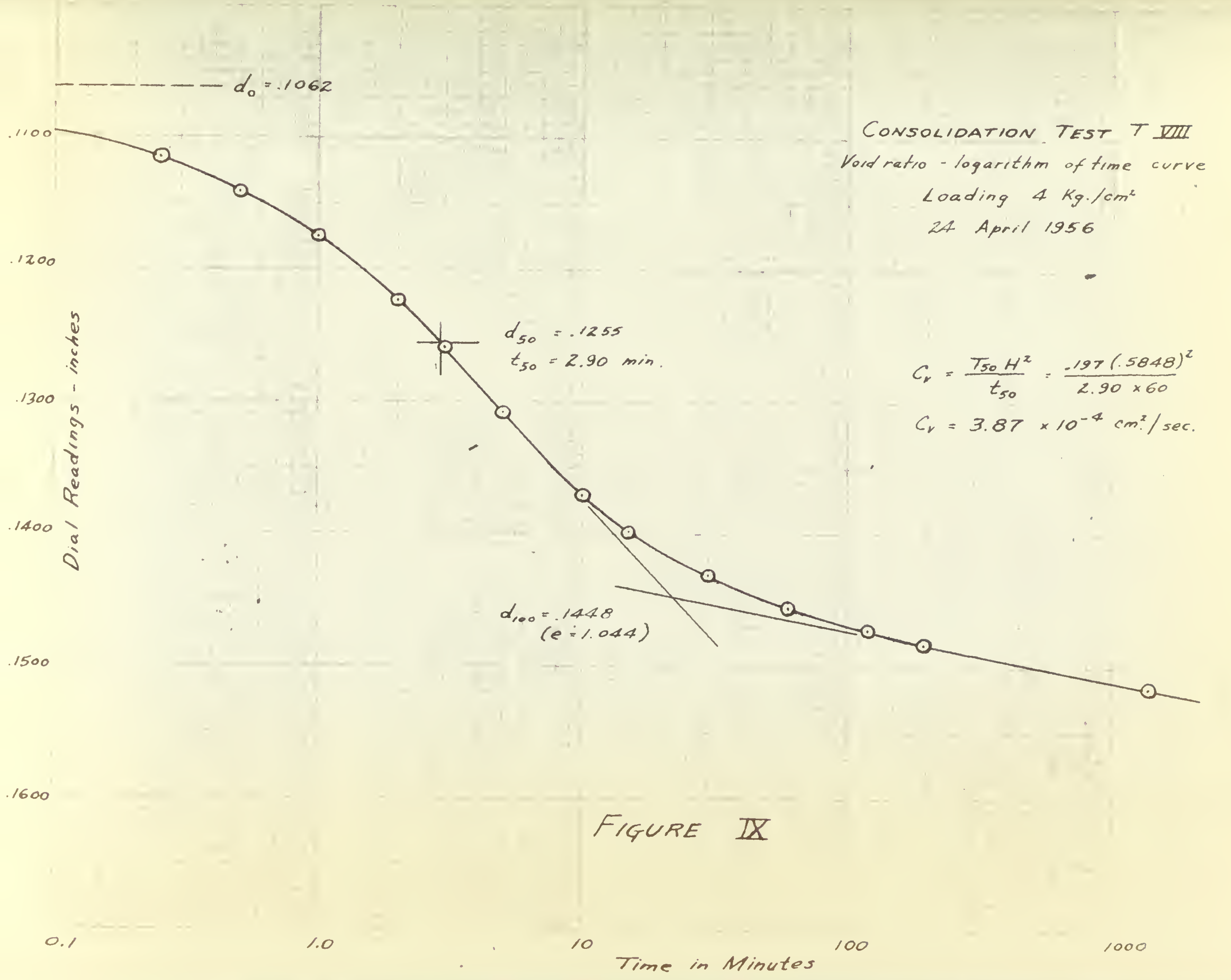

CONSOLIDATION TEST T VIII
Void ratio - logarithm of time curve
Loading 4 Kg./cm²
24 April 1956
$C_v = \frac{T_{50} H^2}{t_{50}} = \frac{.197\,(.5848)^2}{2.90 \times 60}$
$C_v = 3.87 \times 10^{-4}$ cm²/sec.
$d_o = .1062$
$d_{50} = .1255$
$t_{50} = 2.90$ min.
$d_{100} = .1448$
$(e = 1.044)$
.1100
.1200
.1300
.1400
.1500
.1600
Dial Readings - inches
0.1
1.0
10
100
1000
Time in Minutes

FIGURE IX

the time factor. The theoretical curve at 100 per cent consolidation forms an asymptote with the horizontal; thus, on the laboratory curve, 100 per cent consolidation is located at the intersection of the tangent to the curve at the point of inflection, and a straight line produced backward from the lower straight-line portion of the curve. For the loading increment shown in Figure IX, 100 per cent primary consolidation, d_{100}, is at dial reading 0.1448. To determine the zero point of primary consolidation, the upper portion of the curve is considered to follow a parabolic path to the zero per cent primary consolidation. Using the semi-logarithmic plot, this point may be located by marking off the difference in ordinates between 0.1 and 0.4 minutes on the upper portion of the curve, and by plotting this difference above the curve at the time 0.1 minute. By using several points on the upper portion of the curve, in the ratio of 1 to 4, several differences in ordinates may be determined. The average of these ordinate distances was used to determine the zero per cent consolidation. In Figure IX, this zero point is shown as d_0 at the dial reading 0.1062.

By knowing the zero and 100 per cent primary consolidation points on the curve, the 50 per cent consolidation and the time required to reach that point, t_{50}, are determined from the laboratory curve. This data

is then applied to the equation determining the coefficient of consolidation for the particular load increment, as shown in Figure IX. The T_{50} is the time factor for 50 per cent consolidation, and is equal to 0.197. H is equal to one-half the average thickness of the sample during primary consolidation.

A typical laboratory time curve using the Square Root of Time Fitting Method, for the same sample and load increment as in the previous example, is shown in Figure X. Again comparing this laboratory curve with the theoretical curve, plotted with the percentage of consolidation as ordinate and the square root of the time factor as abscissa, there is a similarity in shape. Taylor concluded that, by drawing a straight line through the initial points of the laboratory time curve, as shown in Figure X, the curve at 90 per cent consolidation should be at a distance 1.15 times the abscissa of the straight line.[12]

To find the 90 per cent consolidation point on the laboratory curve, a straight line is drawn through the initial points of the curve, or a straight line which best fits these points is drawn. Another straight line is then drawn to the right, with an abscissa 1.15 times greater than that of the first line. 90 per cent consolidation, d_{90}, is located where the second line crosses the curve. In Figure X, this is at dial reading 0.1358. The zero

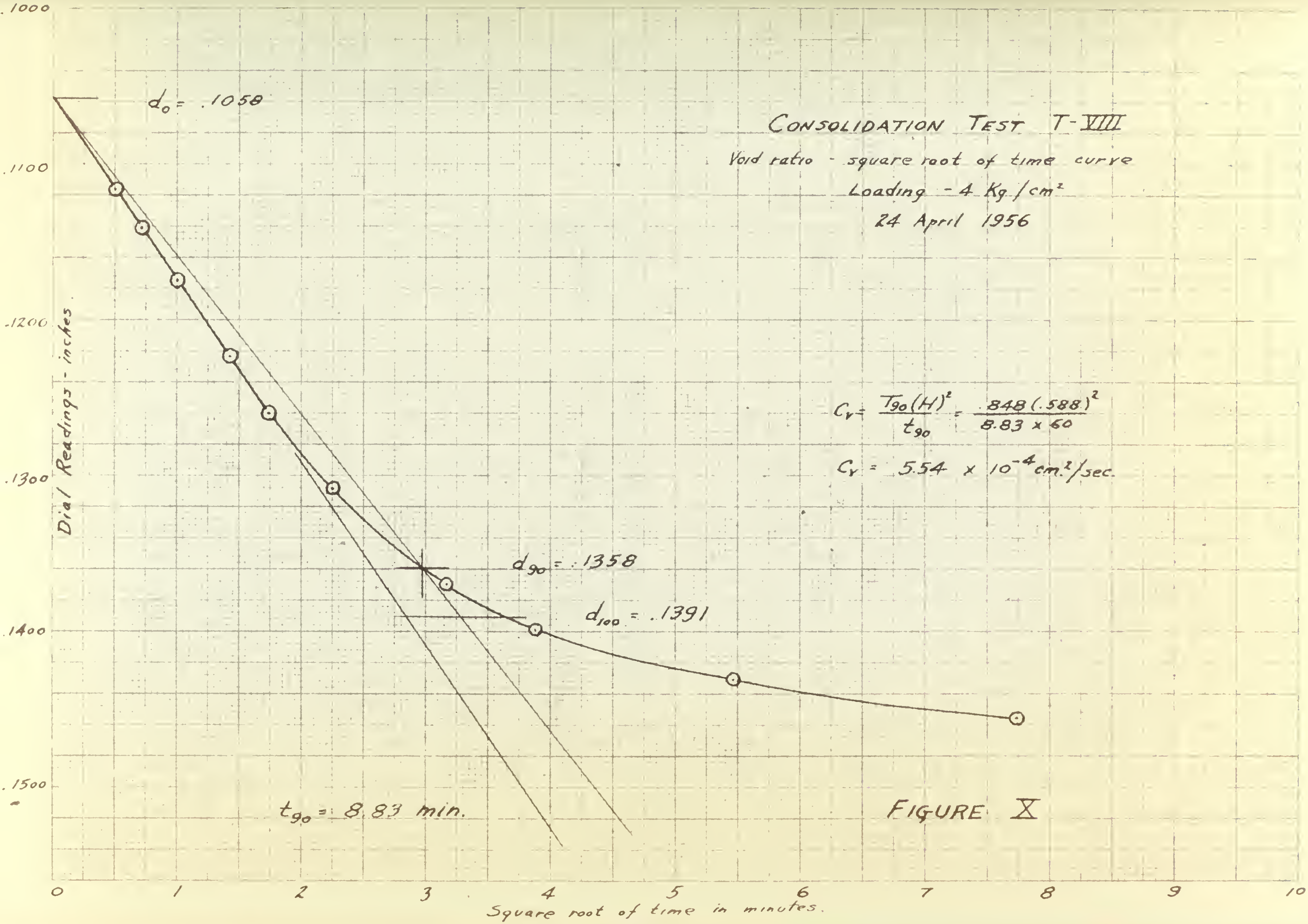

CONSOLIDATION TEST T-VIII
Void ratio - square root of time curve
Loading - 4 Kg./cm²
24 April 1956
$C_v = \frac{T_{90}(H)^2}{t_{90}} = \frac{.848(.588)^2}{8.83 \times 60}$
$C_v = 5.54 \times 10^{-4}$ cm.²/sec.
d_o = .1058
d_{90} = .1358
d_{100} = .1391
t_{90} = 8.83 min.
Dial Readings - inches
.1000
.1100
.1200
.1300
.1400
.1500
Square root of time in minutes.
0
1
2
3
4
5
6
7
8
9
10

FIGURE X

point, d_o, is the point where the straight line, drawn through the initial points on the curve, intersects the zero time which, with this fitting method, is 0.1058.

After locating the 90 per cent point, 100 per cent primary consolidation may be determined easily. With this data, the coefficient of consolidation for the load increment is determined by the equation shown in Figure X. The T_{90} in the equation is the time factor for 90 per cent consolidation and is equal to 0.848. H is equal to one-half the average thickness of the sample during primary consolidation, and t_{90} is the time in seconds at d_{90}.

B. Testing Procedure for Direct Measurement of Permeability Using Air Pressure

With the variable-head permeameter attachment using air pressure to produce higher heads of water, it was possible to run a large number of permeability tests at the end of each loading increment. In one test, permeability measurements were not obtained because small leaks developed around the consolidometer gasket, and the resulting measurements were incorrect. This condition was corrected in all other tests by making certain that the consolidometer assembly bolts were secure. In most of the tests conducted during each loading increment, varying air pressures were used on successive runs to

determine the effect, if any, of various heads on the measured permeability. Again, every precaution was taken to ensure complete saturation of the system from the permeameter standpipe to the lower drainage passages in the consolidometer.

The length of the tests ranged between fifteen to sixty minutes, depending on the magnitude of loading increment, the air pressure used, and the measured drop in head from h_0 to h_1. More lengthy tests and higher pressures were required for the heavier consolidation loads, as expected, because of the lower values of permeability resulting from the reduction in void spaces. One advantage of the attachment used is that the thickness of the sample was accurately known by micrometer dial readings at all times during a permeability test. These readings could be converted to values of void ratio in the sample at the time of testing.

The following procedure was used in performing direct permeability tests for this investigation:

(1) Testing for permeability commenced at the completion of each consolidation loading increment, usually about twenty-four hours after application of the load. Prior to applying head, the micrometer dial reading and time were recorded.

(2) Next, the standpipe in the permeability

attachment was filled with water through the two-way stopcock from the water reservoir. The water in the standpipe was then released slowly by manipulating the stopcock to fill the capillary tube leading to the consolidometer. When the tube was completely full and free from air bubbles, it was connected via a short rubber hose section to the top of the standpipe extending up from the consolidometer base. Excess water in making the connection was permitted to refill the permeameter standpipe.

(3) With the tie-in established between the consolidometer and permeability attachment, both the air pressure regulating valve and air pressure cut-out valve were closed. Next, the main shut-off valve was opened, releasing pressure to the air pressure regulator. The pressure gauge was checked to make certain that no pressure was passing the regulating valve. See Figure VIII.

(4) Next, the air pressure cut-out valve was opened and by a gradual adjustment of the regulating valve, air pressure was slowly applied to the system. While air pressure was being increased, a continuous check was maintained on the level of mercury in the manometer and the height of water in the standpipe. Since the diameter of the standpipe was very small, compression of the entrapped air in the water caused a considerable drop in the standpipe water level. Generally, this drop in

water level was great enough to necessitate refilling the standpipe before a test could commence. This was accomplished by closing the stopcock and air pressure cut-off valve, and releasing the pressure in the permeameter by opening the pressure release valve. Then, by rotating the stopcock, water from the reservoir refilled the standpipe. When the water column was refilled, the stopcock was again closed and pressure was applied to the system by opening the cut-out valve. After this was done the stopcock was opened, allowing flow from the standpipe to the base of the consolidometer.

(5) The process of increasing the pressure and refilling the standpipe by cutting off the air pressure was repeated until correct pressure for the particular test being run was obtained. For loading increments of $\frac{1}{4}$ and $\frac{1}{2}$ Kg. per cm.^{2}, it was necessary to refill the standpipe only once if at all; however, when the sample was under higher consolidation stresses and higher air pressures were necessary to produce a gradual flow through the sample, the process had to be repeated several times.

The increased water pressures will cause a slight expansion of the sample, due to the reduction of intergranular stress from increased neutral pressure and seepage pressure of the upward flow of water. To prevent excessive expansion, the maximum air pressures used for

any one loading increment should not exceed those listed in Table II.

(6) To check the increase in volumn of the sample, readings on the dial micrometer were taken before and immediately upon application of air pressure, as well as approximately fifteen minutes afterward, by which time swelling of the sample usually had ceased. Additional sample thickness readings were taken at the end of the pressure permeability test and after release of the pressure head.

(7) As well as limiting air pressures during the permeability test to those shown in Table II, the pressure should be limited so that it will not cause swelling of the sample in excess of 0.0003 inch during any one loading increment. This limitation may be checked by noting the slight change on the dial micrometer during the building up of air pressure. This is considered necessary to avoid any effect on the consolidation test because of the permeability measurements. Generally speaking, an increase in volume in excess of 0.0003 inch will not occur if air pressures are kept below those listed in Table II.

(8) After air pressure had been applied to the water column for about fifteen minutes and the initial swelling of the sample had occurred, the height of the

TABLE II

Maximum Air Pressures Recommended
for Permeability Measurements
when using the Consolidometer as
A Variable-head Permeameter

Consolidation Loading Pressure in Kg./cm.^{2}	Consolidation Loading Pressure in p.s.i.	Maximum Air Pressures Recommended for Permeability Test		
		p.s.i.	Cm. of Hg.	Percentage of Consolidation Loading
1/4	3.47	.87	4.5	25
1/2	6.94	1.74	9.0	25
1	13.88	3.47	18.0	25
2	27.76	6.94	36.0	25
4	55.5	8.25	43.0	15
8	111.0	11.10	57.4	10
16	222.0	14.44	74.6	6.5

water column, h_o, and time were recorded. At the start and completion of the permeability run, the exact air pressure being applied, h_p, also was measured and recorded from the mercury manometer. These measurements were usually necessary only at the beginning and end of each run; however, if pressure fluctuated they were taken every three minutes, using the average pressure to calculate the total head. Later, when sufficient drop had occurred in the water column, the height, h_1, and time were recorded again. The time interval between heights h_o and h_1 was maintained by a stopwatch. With this information, the coefficient of permeability of the sample was calculated by the formula cited in Part II, Section B-2.

(9) Usually, several permeability tests were run for each load increment. To do this, the stopcock was closed after the h_1 reading, the pressure cut-off valve was closed, and the air pressure release valve was opened, thereby releasing the system from pressure. Then, the standpipe was refilled, the stopcock again closed, and air pressure was applied by closing the release valve and opening the cut-off valve. By opening the stopcock and permitting flow from the standpipe to the consolidometer, the drop in head in the standpipe could again be recorded. In most tests, different heads were used on successive runs. When this was the case, a short time was allowed (but did

not prove sufficient as later shown in the Results) for the sample to expand under the new pressure before permeability head readings were commenced.

(10) When testing for permeability had been completed, the main shut-off valve was closed and air was released from the system. The sample then compressed to its original thickness, or slightly smaller than its measurement at the commencement of testing. With the pressure released, the stopcock was closed and the connection broken between the permeameter standpipe and the base of the consolidometer, again allowing free drainage from the sample in that direction.

In some tests under lighter consolidation loads, sufficient flow of water through the sample could be obtained with the head of water in the standpipe, without the need for additional air pressure. In those tests, a normal variable-head permeability test was conducted with the same apparatus.

PART V

RESULTS AND DISCUSSION

A. General

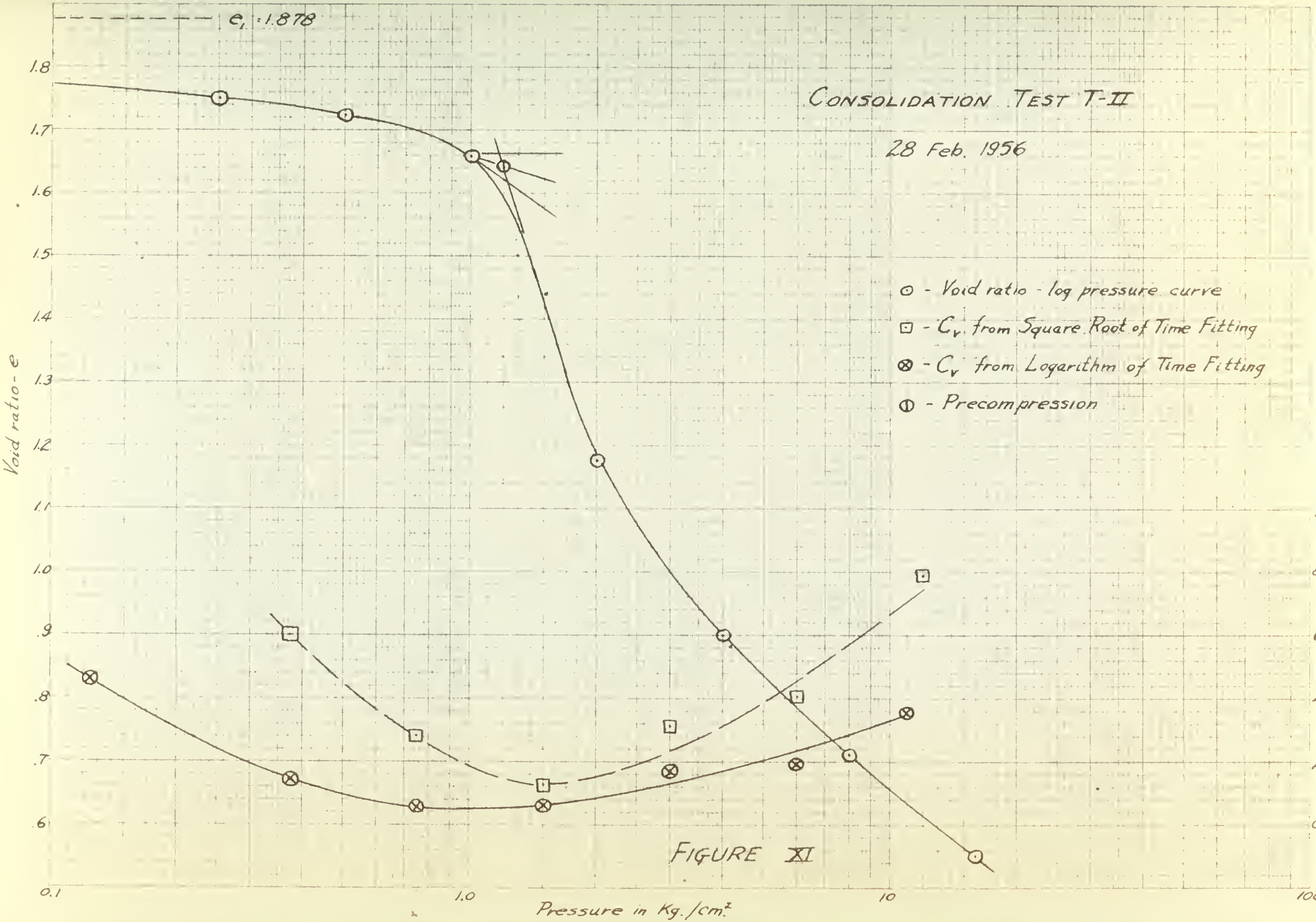
The family of curves resulting from the determination of the coefficient of consolidation by the three methods (1) Logarithm of Time Fitting Method, (2) Square Root of Time Fitting Method, and (3) calculating C_v by using the direct determination of permeability, are plotted for the tests directly below the familiar e - log p curve for each sample. These results are shown in Figures XI, XII, XIV, XVI, XVIII, XX, and XXII. Since direct permeability measurements were erroneous for Test T-II, Figure XI shows only the C_v curves developed from the two fitting methods.

The direct permeability determinations found as a result of the variable-head permeameter tests (with and without the use of air pressure), are plotted as values multiplied by 10^{-8} cm./sec. versus void ratios in Figures XIII, XV, XVII, XIX, XXI, and XXIII, for each of the tests from T-III to T-VIII. Also plotted on each of these graphs is the calculated coefficient of permeability for comparison with the values of the directly measured permeability.

In the six consolidation tests where permeability measurements were obtained at the end of each consolidation loading period, good agreement was found between the directly measured permeability and the calculated theoretical

permeability, thereby checking the theory of consolidation for this factor. It is evident that, in the later Tests T-VI, T-VII, and T-VIII, closer agreement was obtained between the k_v and k_d determinations. This improvement resulted from the fact that a slightly longer period of time was allowed, in these later tests, between pressure application and actual testing.

The tabulation of laboratory data for each test conducted for both the consolidation and permeability determinations is assembled in the Appendices attached hereto.



CONSOLIDATION TEST T-II
28 Feb. 1956
e₁ = 1.878
⊙ - Void ratio - log pressure curve
⊡ - Cv from Square Root of Time Fitting
⊗ - Cv from Logarithm of Time Fitting
⦶ - Precompression
Void ratio - e
1.8
1.7
1.6
1.5
1.4
1.3
1.2
1.1
1.0
.9
.8
.7
.6
Cv x 10^-4 cm²/sec.
10
8
6
4
2
0
0.1
1.0
10
100
Pressure in Kg./cm²

FIGURE XI

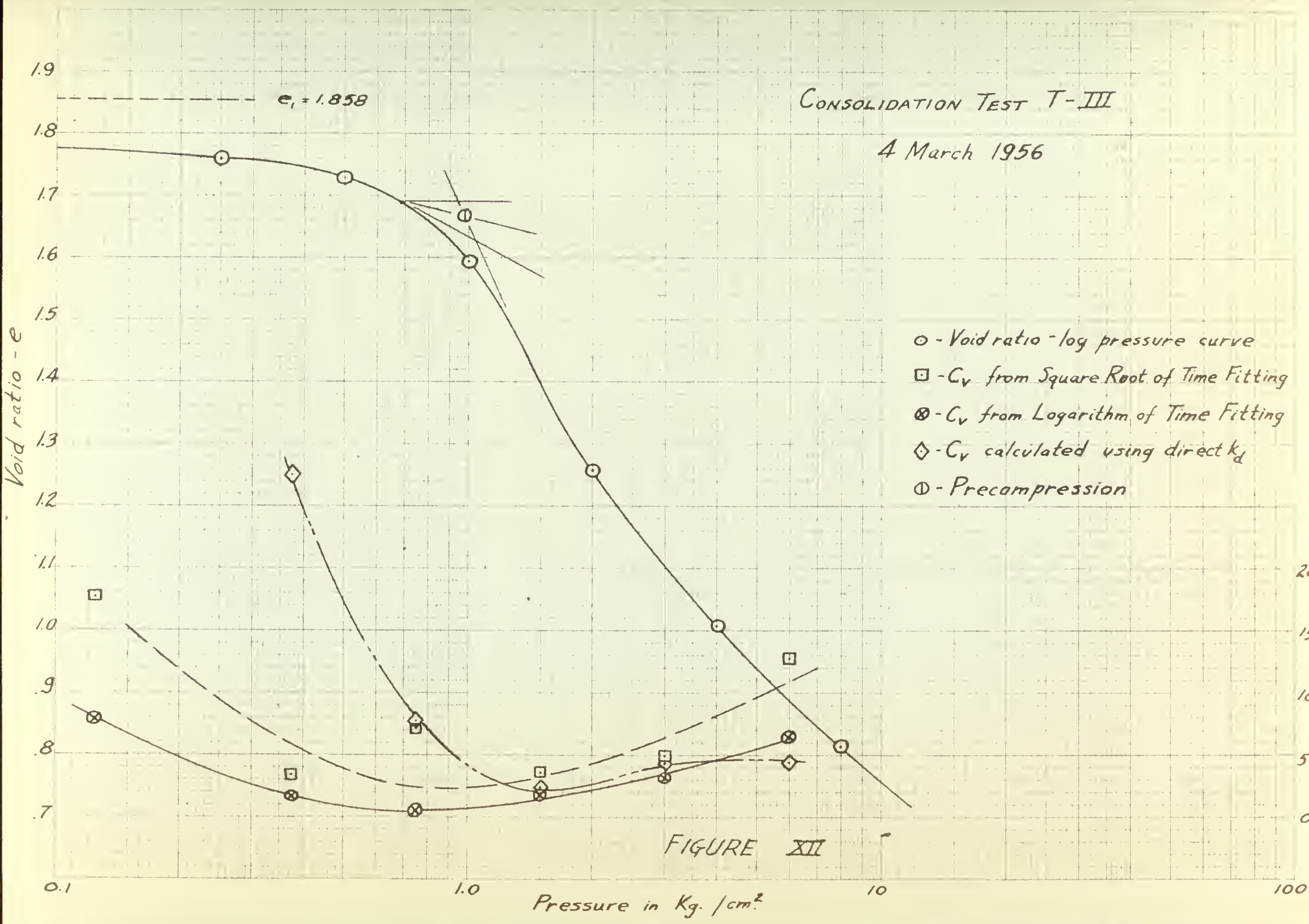

Consolidation Test T-III
4 March 1956
$e_1 = 1.858$
○ - Void ratio - log pressure curve
□ - C_v from Square Root of Time Fitting
⊗ - C_v from Logarithm of Time Fitting
◇ - C_v calculated using direct k_d
⦶ - Precompression
Void ratio - e
1.9
1.8
1.7
1.6
1.5
1.4
1.3
1.2
1.1
1.0
.9
.8
.7
$C_v \times 10^{-4}$ cm²/sec.
20
15
10
5
0
0.1
1.0
10
100
Pressure in Kg. /cm²

FIGURE XII

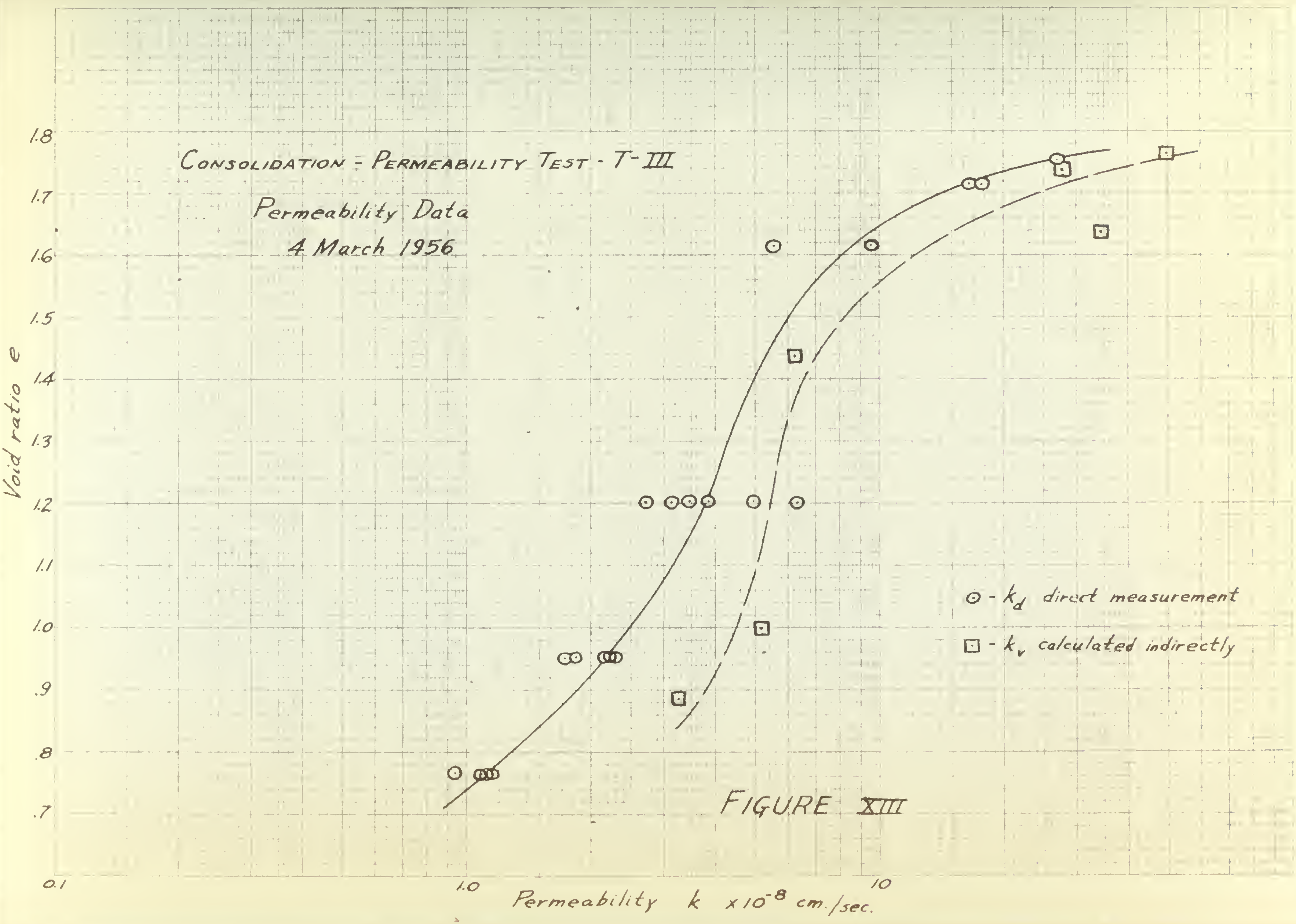

Consolidation - Permeability Test - T-III
Permeability Data
4 March 1956
1.8
1.7
1.6
1.5
1.4
1.3
1.2
1.1
1.0
.9
.8
.7
Void ratio e
0.1
1.0
10
Permeability k x10^-8 cm./sec.
⊙ - k_d direct measurement
⊡ - k_v calculated indirectly

FIGURE XIII

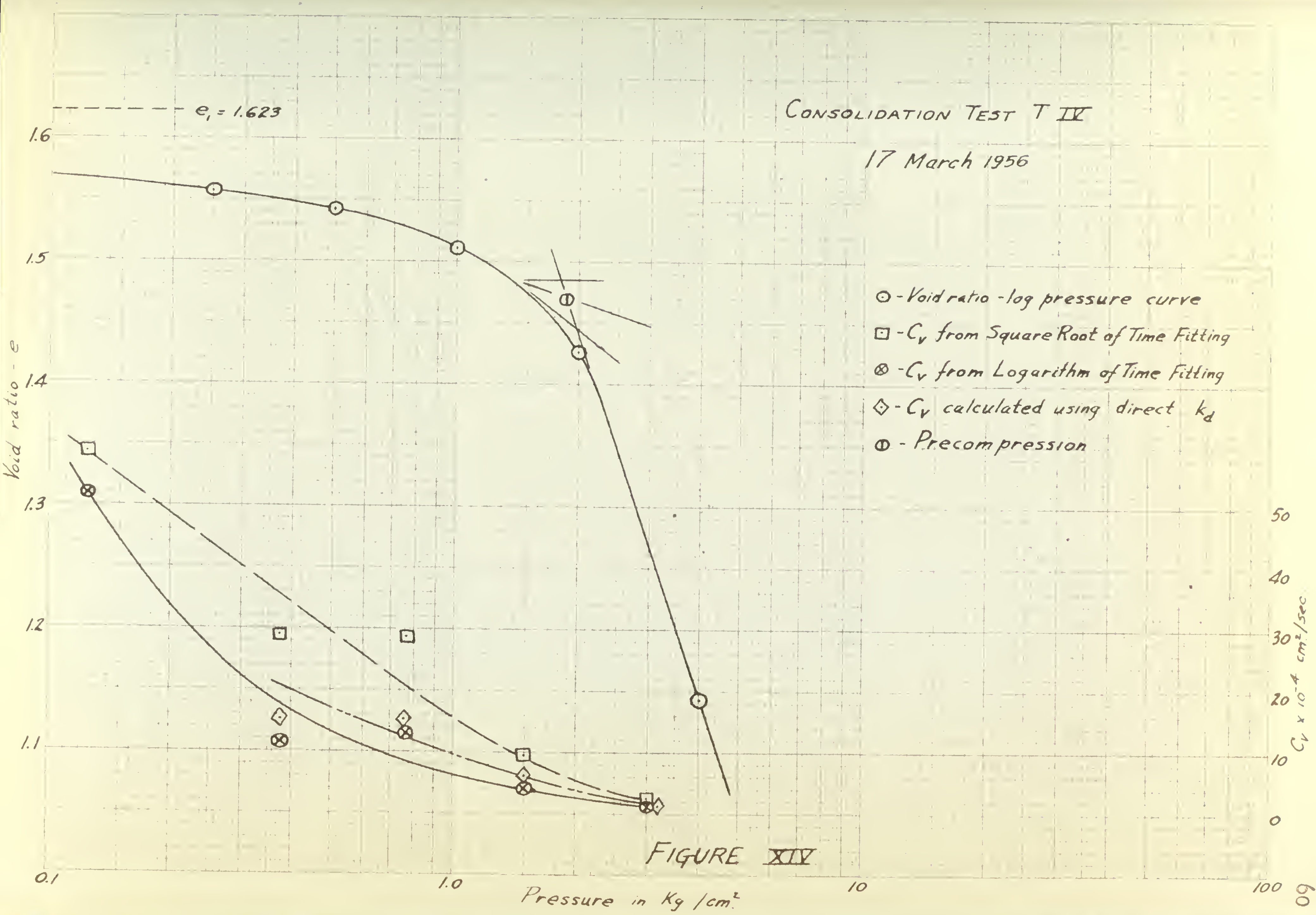
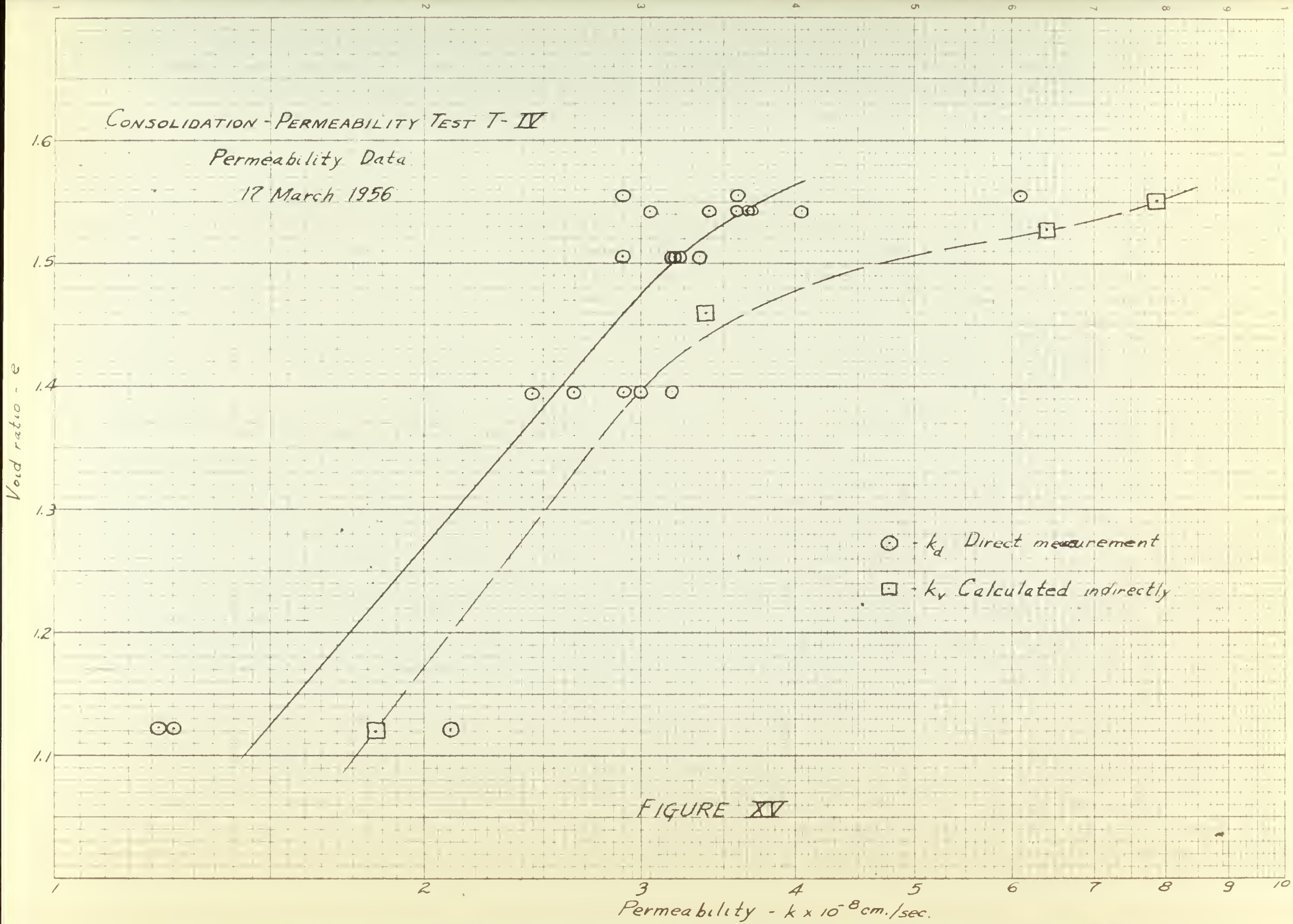
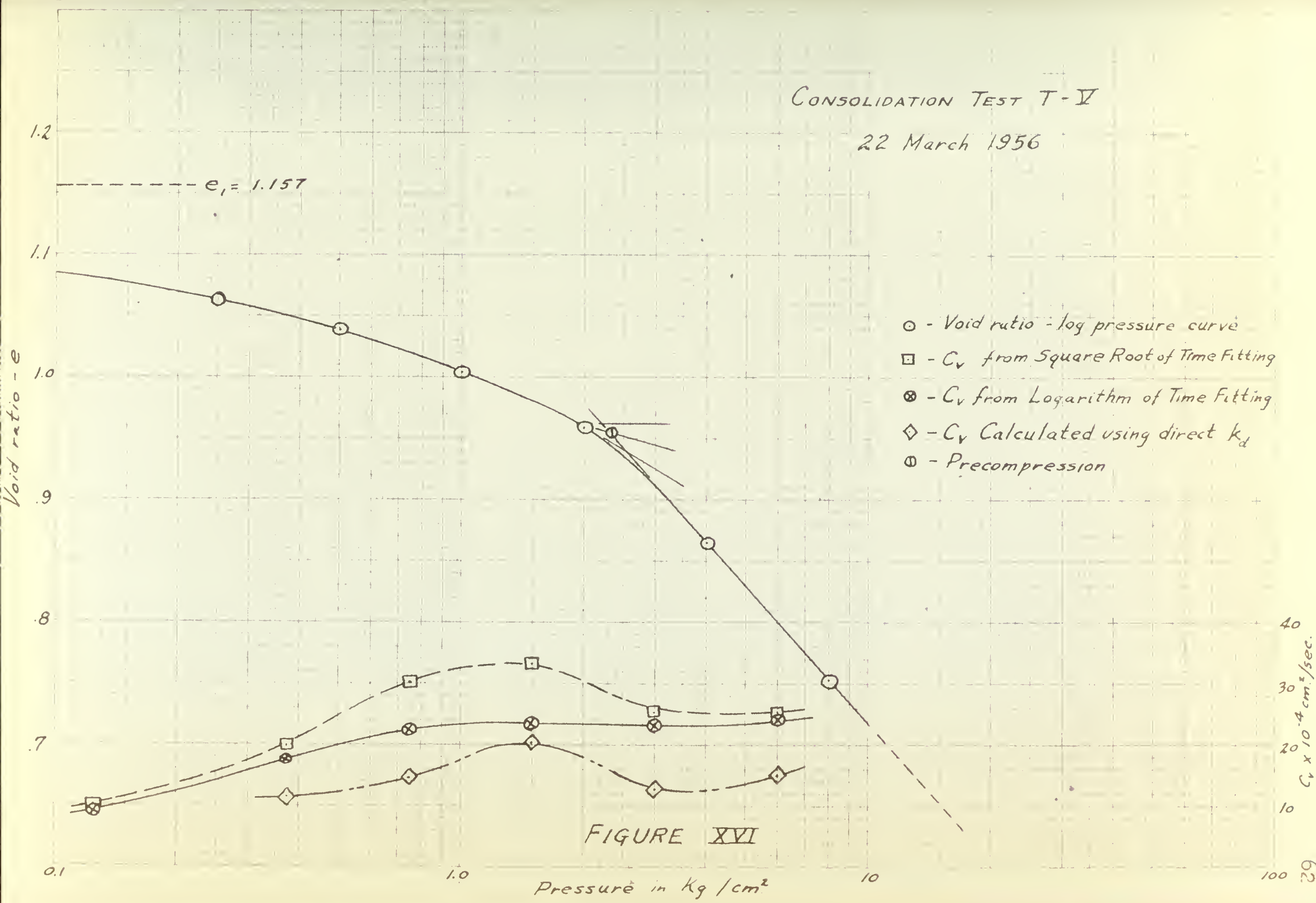

Consolidation Test T IV
17 March 1956
e₁ = 1.623
- Void ratio - log pressure curve
- C_v from Square Root of Time Fitting
- C_v from Logarithm of Time Fitting
- C_v calculated using direct k_d
- Precompression
Void ratio - e
1.6
1.5
1.4
1.3
1.2
1.1
0.1
1.0
10
100
Pressure in Kg /cm²
$C_v \times 10^{-4}$ cm²/sec
50
40
30
20
10
0

FIGURE XIV



Consolidation - Permeability Test T-IV
Permeability Data
17 March 1956
⊙ - k_d Direct measurement
⊡ - k_v Calculated indirectly
Void ratio - e
1.6
1.5
1.4
1.3
1.2
1.1
1
2
3
4
5
6
7
8
9
10
Permeability - k x 10^{-8} cm./sec.

FIGURE XV



CONSOLIDATION TEST T-V
22 March 1956
e₁ = 1.157
⊙ - Void ratio - log pressure curve
⊡ - Cv from Square Root of Time Fitting
⊗ - Cv from Logarithm of Time Fitting
◇ - Cv Calculated using direct kd
⦶ - Precompression
1.2
1.1
1.0
.9
.8
.7
Void ratio - e
0.1
1.0
10
100
Pressure in Kg/cm²
40
30
20
10
Cv x 10⁻⁴ cm²/sec.

FIGURE XVI

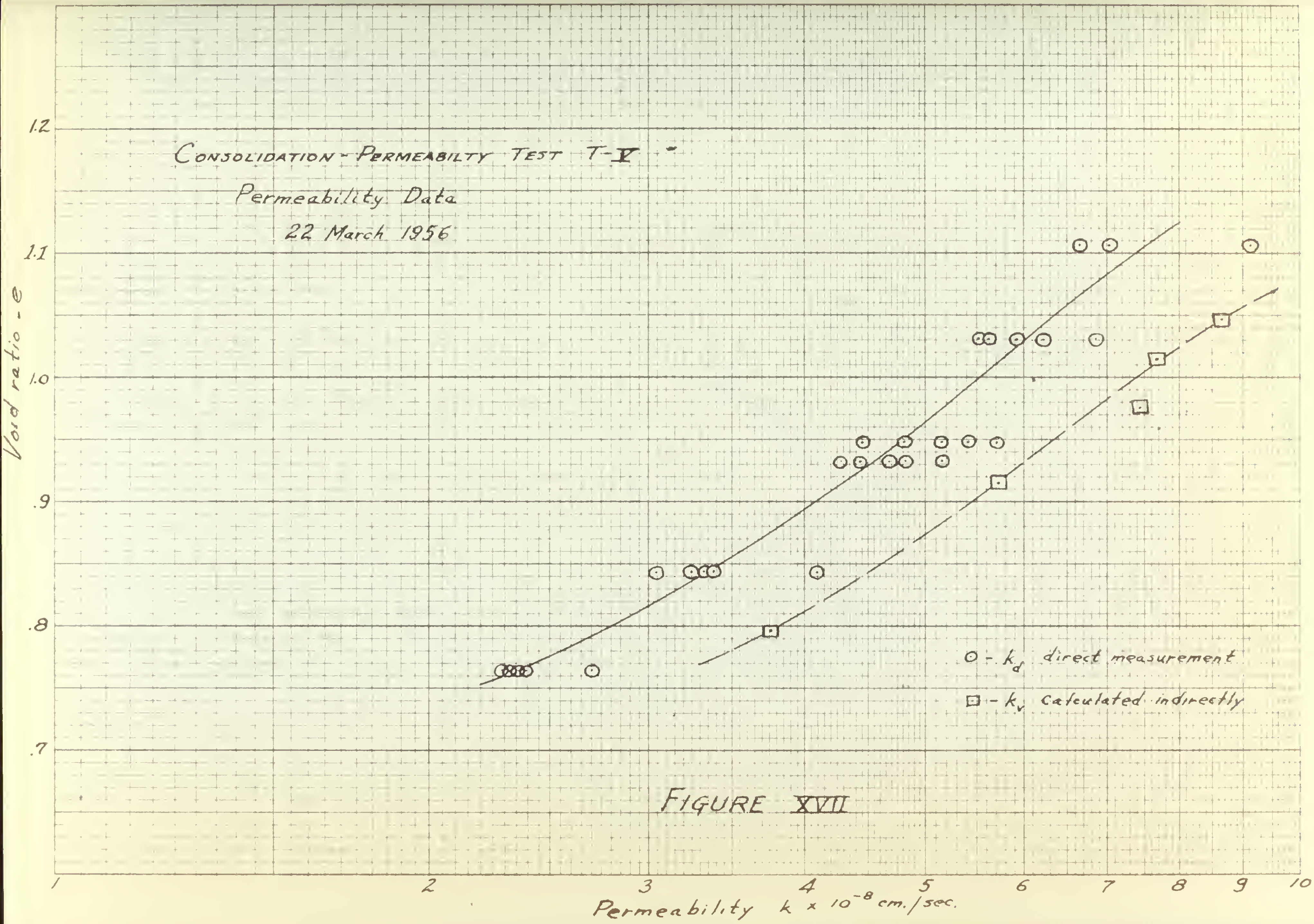

Consolidation - Permeabilty Test T-V
Permeability Data
22 March 1956
1.2
1.1
1.0
.9
.8
.7
Void ratio - e
⊙ - k_d direct measurement
⊡ - k_v calculated indirectly
1
2
3
4
5
6
7
8
9
10
Permeability $k \times 10^{-8}$ cm./sec.

FIGURE XVII

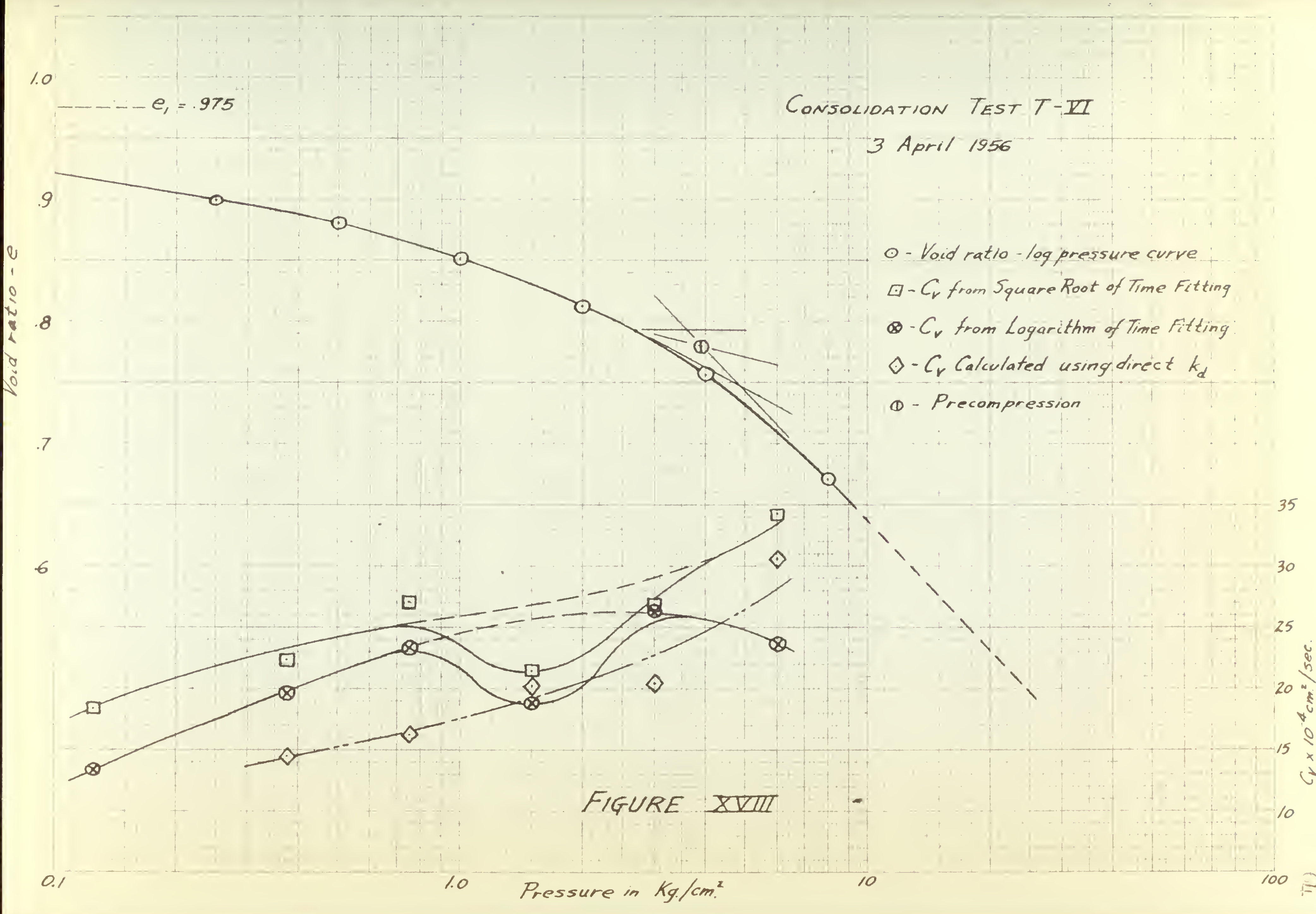
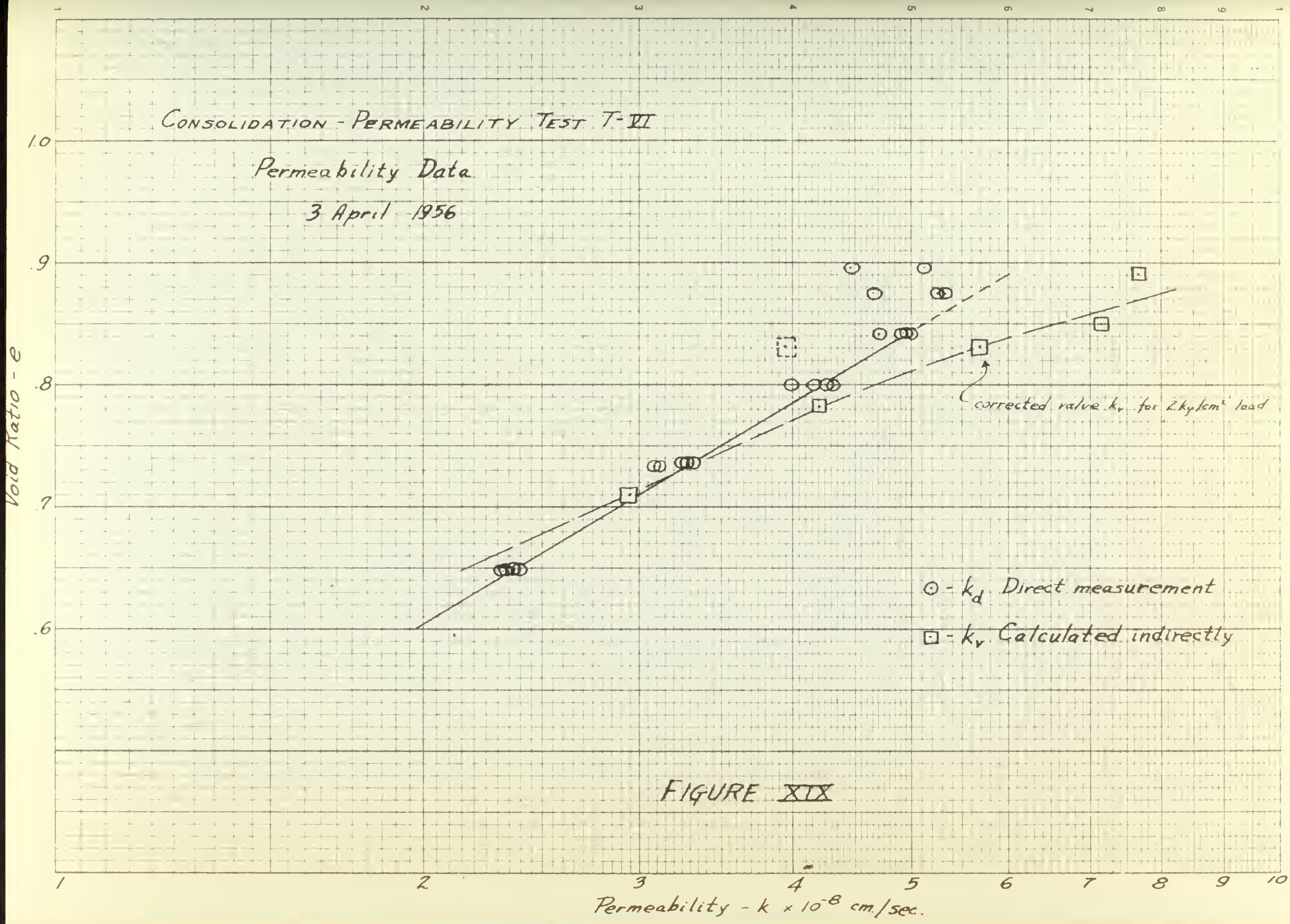


FIGURE XVIII


CONSOLIDATION - PERMEABILITY TEST T-VI
Permeability Data
3 April 1956
corrected value k_v for 2kg/cm² load
⊙ - k_d Direct measurement
⊡ - k_v Calculated indirectly
FIGURE XIX
Void Ratio - e
1.0
.9
.8
.7
.6
1
2
3
4
5
6
7
8
9
10
Permeability - k × 10^{-8} cm./sec.
55

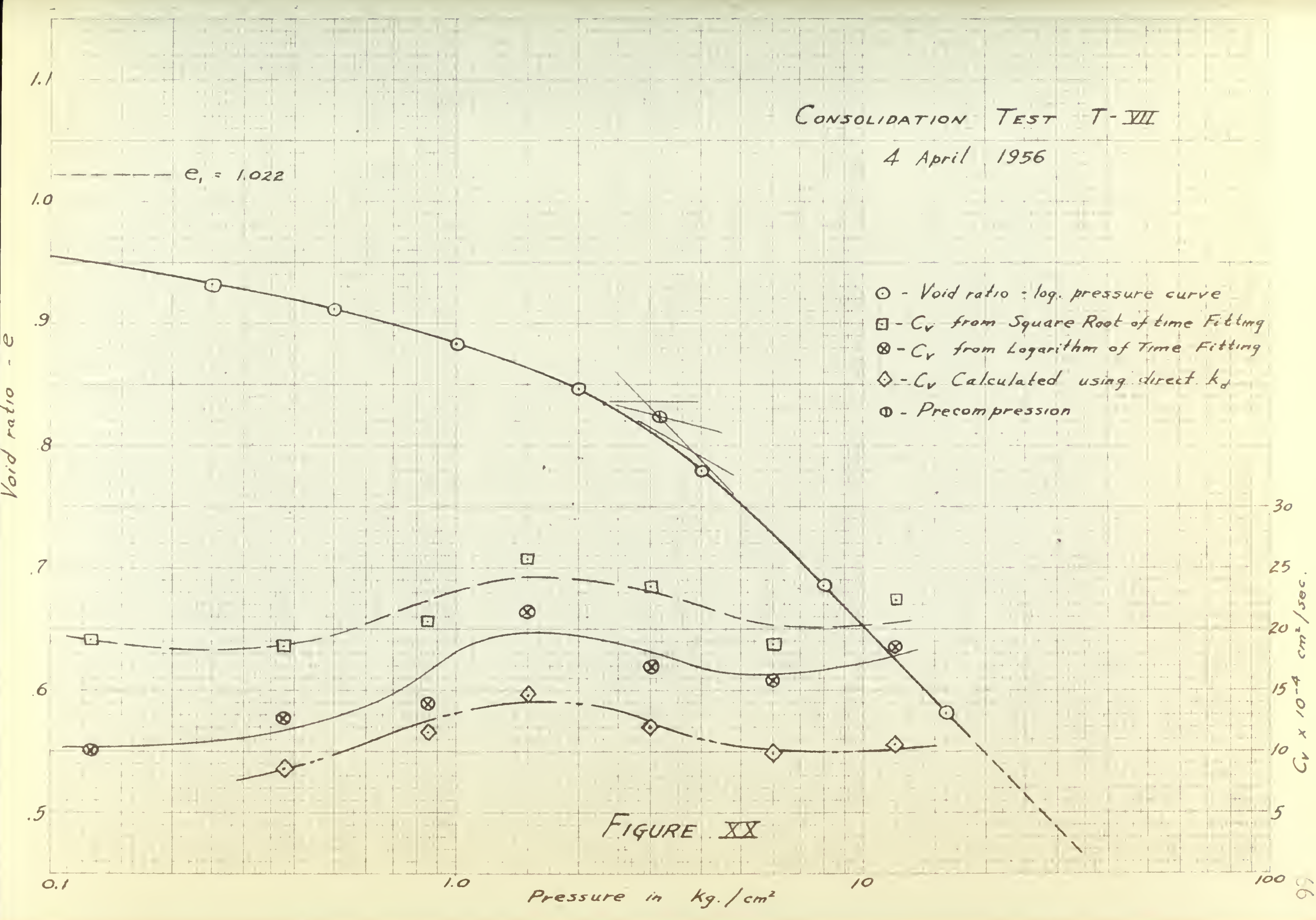

CONSOLIDATION TEST T-VII
4 April 1956
e_1 = 1.022
⊙ - Void ratio : log. pressure curve
⊡ - C_v from Square Root of time Fitting
⊗ - C_v from Logarithm of Time Fitting
◇ - C_v Calculated using direct k_d
⦶ - Precompression
Void ratio - e
1.1
1.0
.9
.8
.7
.6
.5
30
25
20
15
10
5
$C_v \times 10^{-4}$ cm²/sec.
0.1
1.0
10
100
Pressure in kg./cm²

FIGURE XX

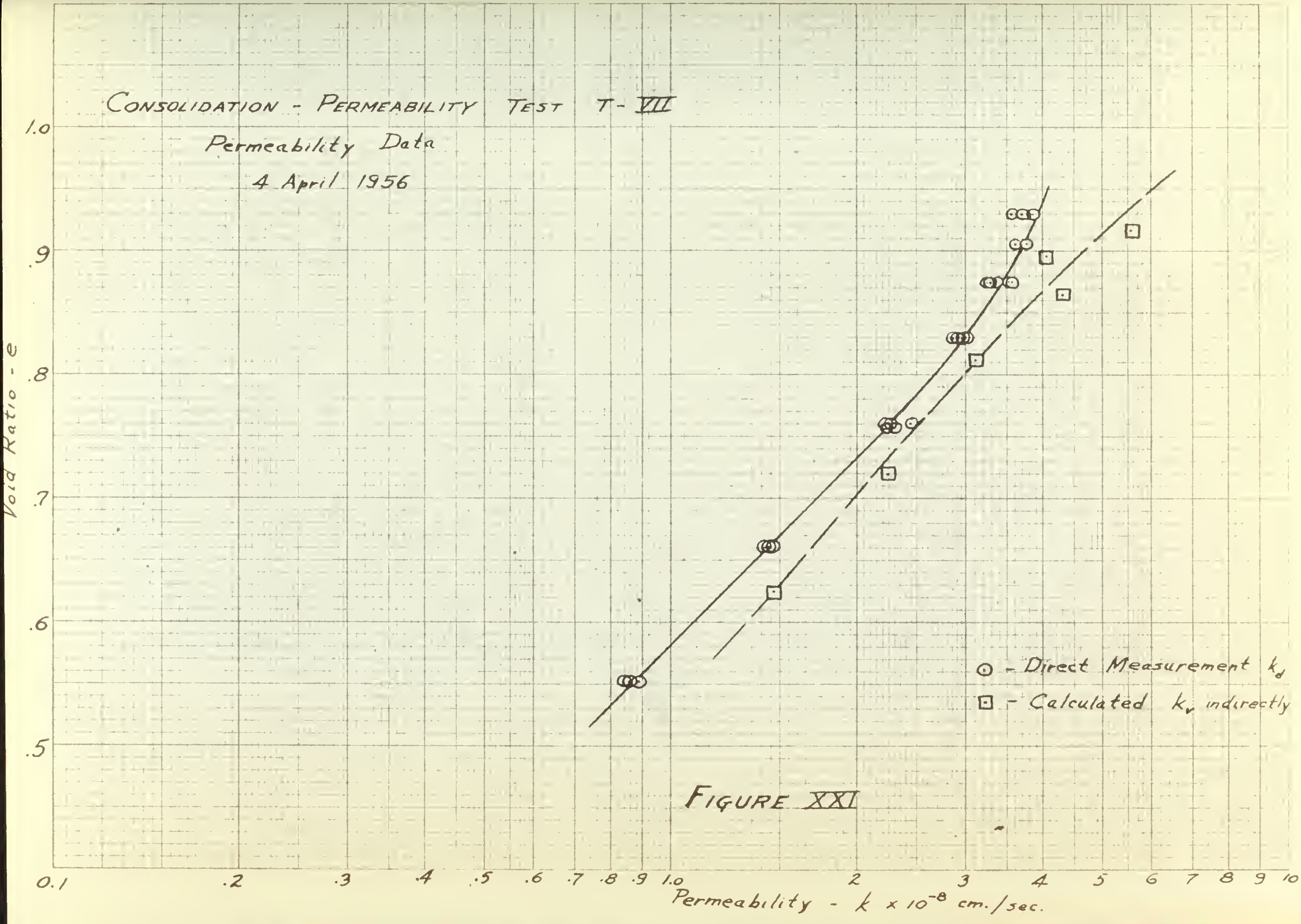

CONSOLIDATION - PERMEABILITY TEST T-VII
Permeability Data
4 April 1956
Void Ratio - e
1.0
.9
.8
.7
.6
.5
0.1
.2
.3
.4
.5
.6
.7
.8
.9
1.0
2
3
4
5
6
7
8
9
10
Permeability - k x 10⁻⁸ cm./sec.
⊙ - Direct Measurement k_d
⊡ - Calculated k_v indirectly

FIGURE XXI

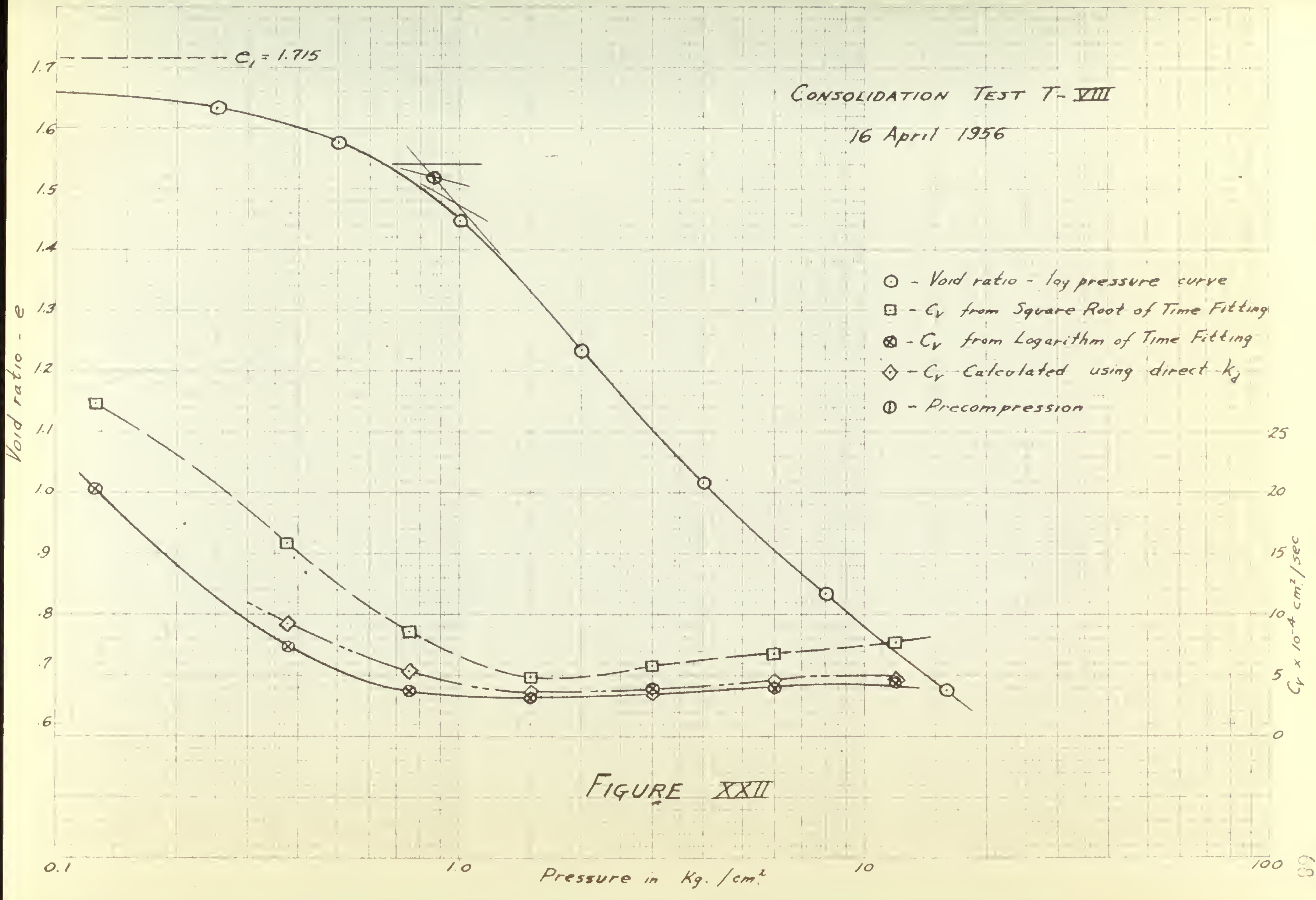

Consolidation Test T-VIII
16 April 1956
$e_1 = 1.715$
⊙ - Void ratio - log pressure curve
⊡ - C_v from Square Root of Time Fitting
⊗ - C_v from Logarithm of Time Fitting
◇ - C_v Calculated using direct k_d
⦶ - Precompression
Void ratio - e
1.7
1.6
1.5
1.4
1.3
1.2
1.1
1.0
.9
.8
.7
.6
$C_v \times 10^{-4}$ cm²/sec
25
20
15
10
5
0
0.1
1.0
10
100
Pressure in Kg./cm²

Figure XXII

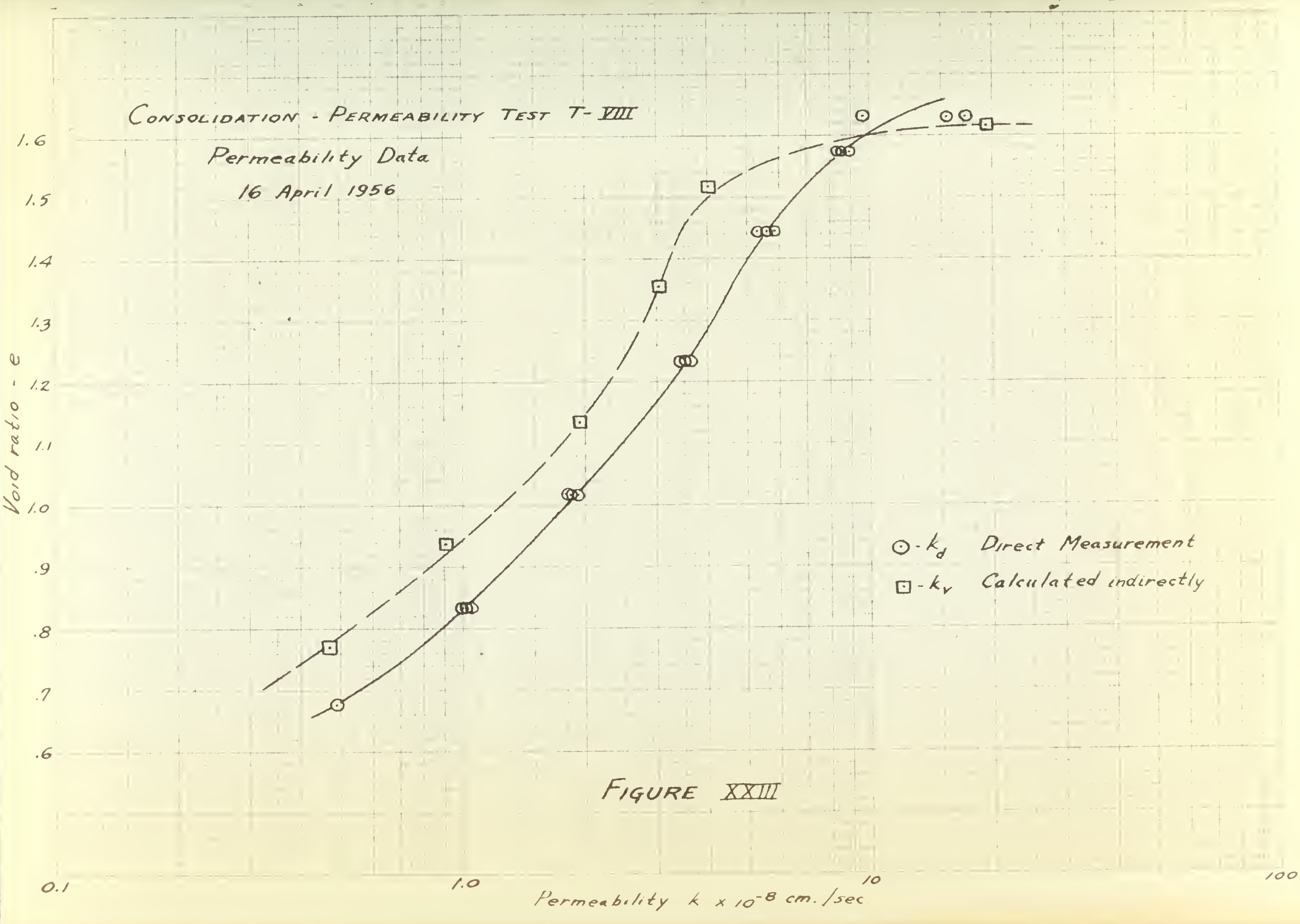


FIGURE XXIII

B. Shape of the Resulting C_v Curves

In the Figures cited previously in this Part, the coefficient of consolidation, C_v, is plotted against the logarithm of pressure values at the mean pressure for each of the loading increments. Since this soil property is evaluated from the average values of the loading increment, plotting C_v at the mean increment pressure is considered accurate and is commonly used.

For all samples tested, the C_v resulting from applying the Square Root of Time Fitting Method to the time curves for each loading increment were higher than the C_v resulting from use of the Logarithm of Time Fitting Method. The separate values of C_v compared more closely with one another near the precompression range of the particular sample tested. To determine which of these fitting methods provides the most accurate value for determining this soil property is not within the scope of this investigation. However, by applying the directly measured permeability coefficient to the theoretical formula for finding C_v, it was found that this plotted value of C_v more closely approximated the values of C_v found by the Logarithm of Time Fitting Method. This was not true for all test results but, generally speaking, for the limited number of tests conducted, the values of C_v obtained by Casagrande's Logarithm of Time Fitting Method

were more closely related to the calculated value of C_v utilizing the direct permeability determinations. To relate the value of the two fitting methods to actual settlement predictions, a great deal more study together with accurate time-settlement measurements of structures erected over compressible clay stratas will have to be accomplished.

The changing shape of the C_v curve, obtained from determinations using the two fitting methods, is similar throughout each consolidation test. The shape of the C_v curve developed indirectly through use of the direct permeability determinations also followed the general shape of the other two C_v curves, except in Test T-VI.

Most of the C_v curves for the samples tested do show a tendency to digress, resulting in a maximum or minimum ordinate in the vicinity of the pressure determined as the precompression in the sample. This change in C_v value, however, is gradual and is usually not distinct enough to evaluate precompression. This relationship is shown in the family of curves plotted for C_v against pressure in Figures XI, XII, XIV, XVI, XVIII, XX, and XXII. Although this changing value of C_v does not provide an accurate means of measuring precompression, the change is noticeable and appears on the curve in the vicinity of the sharp bend in the void ratio-logarithm of pressure curve.

Figure XVIII shows the test results for sample T-VI, which includes the three separate C_v curves. It is apparent that, for the 2 Kg./cm.^{2} loading increment, the C_v curve for the two fitting methods does not follow the general pattern of the C_v curve resulting from direct measurements of permeability. To further check this discontinuity, the calculated coefficient of permeability, k_v, for that loading increment, plotted out of line with the rest of the calculated values for k_v (See Figure XIX). In this Figure, the indirectly calculated value of k_v for the 2 Kg./cm.^{2} loading increment is shown as a point outlined by a dotted square. Since it plotted out of line with the other values of k_v and the directly measured k_d values, it is believed that an error was thus indicated in the value of C_v for that particular loading increment. To correct the irregularity in the values of C_v, the two curves obtained with the fitting methods were redrawn as smooth curves, disregarding the value for the 2 Kg./cm.^{2} loading. The revised curves are shown as dotted lines in Figure XVIII. Consequently, a new value of C_v for that loading increment, taken from the revised C_v curve, was then used to calculate the indirect value of permeability (See Figure XIX). It was immediately apparent that the correction was warranted, since the corrected value of k_v plotted in line with the other values, approximating a straight line which closely resembled the directly measured permeability, k_d, curve shown also in Figure XIX.

C. Relationship between Direct and Indirect Measurements of Permeability

All of the permeability determinations of direct measurement found between loading increments are plotted in Figures XIII, XV, XVII, XIX, XXI, and XXIII. The average value is shown as a curve representing the direct measurement of permeability in the sample. An appreciable scattering of results is noticeable in the first four tests (T-11 through T-V), and limited scattering is evident in the later tests, except those under the smaller pressure increments. An improvement in obtaining results was reached by allowing more time for the initial swelling of the sample to occur before taking the actual drop in head measurements. This slight change in procedure, allowing approximately fifteen minutes for expansion, provided enough time for upward flow to establish itself throughout the sample, resulting in more accurate readings. In the last test (T-VIII), results of which are shown in Figure XXI, the scattering is as small as can be expected for this type of test.

During the permeability test for any one loading increment, a general tendency was noted for the value of permeability obtained to increase slightly as the pore water pressure increased. This also caused a scattering of results. The variation was probably due to the changing

volume of entrapped air effecting the permeability of the sample tested. Since all undisturbed soil materials will have a certain amount of undissolved gas trapped within the void spaces, a variation in the neutral pressure will have a decided effect on the volume of entrapped gas which, in turn, will effect the size and number of drainage passages available for the flow of water. This effect will have a tendency to increase permeability as the pore water pressure is increased, which actually occurred in the testing. Since the values of permeability obtained during any loading increment did not show a tendency to decrease, from one test to another, it is believed that flow through the sample does not cause an appreciable movement of particles which might block drainage passages or migration of gas bubbles blocking the normal pore channels. For a blocking of flow passages to occur, it is believed that much higher pressures than those used in this study, for the type of samples tested, would have to be used. In view of this fact, it is concluded that a major factor effecting the permeability in the samples tested, when alterations in pressure were used, was the change in volume of the entrapped gas.

A reasonable agreement was evident between the values of the direct and indirect permeability measurements, particularly for the later tests where results

obtained were more accurate. It is believed that this shows that the method employed in preparing the sample and the use of the variable-head permeameter with air pressure provides an accurate means of checking the vertical permeability in relatively impervious soils. This agreement also shows that the movement of water taking place during consolidation is accurately represented by the consolidation theory. The closest relationship between the direct and indirect determinations was obtained in Tests T-VII and T-VIII, shown in Figures XXI and XXIII, with the similarity in curves being most apparent in Figure XXIII. There, the directly measured permeability was plotted as a smooth curve, rather than as the usual approach to a straight line, and the indirect determinations of permeability plotted on the same graph resulted in a similar curve. The similarity in permeability variation throughout most of the change in void ratio clearly shows the agreement between the theoretical value of k and the value which can be measured during consolidation.

Other investigators have found that, generally, a plot of the void ratio versus the logarithm of permeability approximates a straight line.[9, 12] A straight-line plot on such a semi-logarithmic scale would mean that, during the changes in higher values of void ratio, the change in permeability would be greatest, and the rate

of reduction in the value of permeability would decrease as consolidation progressed. The above relationship was found to hold true, to a degree, in all tests. If the permeability determinations did not plot in a straight line, they did approximate a straight line, particularly for the changes in void ratio below the precompression pressure found in the samples tested.

Test T-IV was run with higher than normal pressures for the determination of permeability in order to note the effect of increased pore pressure on the resulting values of permeability. Evidently the high pressure of 52.2 centimeters of mercury (equivalent to 18.5 per cent of the consolidation load) at the 4 Kg./cm.^{2} load increment was too great for the size of sample being used, since the sample ruptured on one edge during the test. This rupture created a hole of about one-eighth of an inch in size in the sample from bottom to top surfaces. Results of this test are shown, however (Figures XIV and XV), up to the point of rupture. Much scattering is evident in the permeability tests for this sample, as shown in the direct permeability results plotted in Figure XV. Again, the scattering is probably due to the effect of sample swelling at the beginning of the test and to changes in the volume of entrapped gas created by variations in pore water pressure.

In Test T-VIII, methylene blue powder was added to the distilled water below the sample and in the permeameter attachment to determine if the values of direct permeability measurements were being effected by the upward flow of water around the edges of the sample. If leakage occurred around the sample edges, it would be indicated by a discoloration of the soil particles resulting from the adsorption of methylene blue.

When the consolidation and permeability tests using the dyed water were completed, it was found that the methylene blue had penetrated and had been adsorbed evenly along the bottom face of the sample to a depth of approximately one-sixteenth of an inch. This indicated that percolation of water was evenly distributed through the sample, that flow was occurring through the sample, and that no appreciable leakage was taking place around the edges. The slight penetration of the dye into the sample was caused by the high ion exchange capacity of the clay minerals, resulting in the complete removal of the methylene blue in the lower layer of the sample. If a high concentration of dye were used, or a greater quantity of water was forced through the sample than that occurring during the permeability tests, the depth of discoloration would have been greater.

Both of the samples for Tests T-VI and T-VII,

results of which are shown in Figures XVIII, XIX, XX, and XXI, were taken from the same tube, one sample removed directly above the other. The two resulting e - log p curves are similar in shape and, if plotted one above the other, would be nearly identical except for the void ratio scale. For Test T-VI the initial void ratio is 0.975 and for Test T-VII it is 1.022. A comparison of the directly measured permeability of the two samples showed that, for the same void ratio, the permeability for sample T-VI was approximately twice that of sample T-VII. Considering the various factors which influence the permeability in a very fine-grained soil, such as composition and structure of soil particles, this difference in values is not considered unusual. In addition to the factors inherent in the soil which effect the value of permeability, no doubt the disturbance of the sample during the sizing operation and placing it in the consolidometer will effect the permeability of the material to a small degree.

The determination of precompression for the two tests showed vide variation mainly because of the very gradual e - log p curves, which made it difficult to select the point of maximum curvature. This initial step in Casagrande's method for determining precompression, when used with test results similar to T-VI and T-VII, shows the need for better methods of determining precompression.

PART VI

CONCLUSIONS

(1) From the limited number of consolidation tests conducted during this investigation, results indicated that the shape of the coefficient of consolidation curve, plotted against the mean logarithm of pressure values, does not follow any set pattern. The break in the curve appears in some tests to occur at or near the estimated precompression of the material. However, there does not appear to be a distinct relationship between the C_v curve and the familiar e - log p curve.

(2) The C_v curves developed by the use of three methods in this study showed a similarity in shape for each sample tested. The values of C_v determined by the Logarithm of Time Fitting Method most closely approximated the values of C_v determined indirectly, using the value of permeability in the sample which was measured by actual tests during the consolidation process.

(3) Throughout each consolidation test, the values of C_v derived from the Square Root of Time Fitting Method were higher than the values determined by the Logarithm of Time Fitting Method. The difference in results was not great in most tests, but this variation in use of the two fitting methods was indicated.

(4) Test results showed that, by following

careful operating procedures, the air-pressure variable-head permeameter provides a relatively accurate means of determing permeability for samples under consolidation loads in the laboratory. The use of air pressure for this purpose is warranted. Disadvantages of the additional apparatus are more than outweighed by the adaptability of the attachment and the accuracy of results obtained from its use.

(5) Direct permeability determinations obtained with the permeameter attachment in this investigation checked closely with the indirect calculations of permeability at various void ratios, when certain precautions were used in the testing procedure. These precautions are outlined in Part IV-B.

(6) The accuracy of the results of directly determined permeability is greatly influenced by the initial swelling of the sample and, unless sufficient time is allowed under a particular pore water pressure for flow through the sample to be established, a great deal of scattering in the plotted results will appear. For best results, the pore water pressure should be held constant for a series of permeability tests at any one loading increment. Fifteen minutes seemed adequate time for the initial swelling to occur, considering the apparatus used and the size of samples tested.

(7) A disadvantage in use of the variable-head permeameter is that the volume of gas trapped within the sample will change because of a decrease in head, during the test. This will, in turn, influence the accuracy of results obtained. By employing air pressure, the change in pressure from one test to the next was high and, no doubt, did effect the results to a small extent. Since the increased pressures were necessary to produce flow under heavier consolidation loads within a reasonable time, it would be difficult to avoid the effect of entrapped gas volume changes on the permeability determinations.

(8) By comparing the values of directly determined permeability with the theoretically calculated value of permeability, it is possible to correct certain values of C_v which do not plot in agreement with the general pattern of the other C_v determinations during the consolidation process.

PART VII

LITERATURE CITED

1. Burmister, D. M., An Application of Controlled Test Methods in Consolidation Testing - Symposium on Consolidation Testing of Soils, A.S.T.M., June 1951.

2. Casagrande, A., The Determination of the Preconsolidation Load and Its Practical Significance - Proc. 3rd Int. Conf. on Soil Mechanics, Vol. III, 1936.

3. Casagrande, A., The Structure of Clay and Its Importance in Foundation Engineering - Journal of the Boston Society of Civil Engineers, Vol. XXVIII, No. 3, July 1941.

4. Cooper, P. L. and Cassie, W. F., The Mechanics of Engineering Soils - McGraw - Hill Book Co., Inc., 2nd Ed., 1953.

5. Darcy, H., Les Fontains Publiques de la Ville de Dijon, Paris - Dijon, 1856.

6. Gilboy, G., Improved Soil Testing Methods - Engineering News Record, May 21, 1936.

7. Gilboy, G., Soil Mechanics Research - Transactions of American Society of Civil Engineers, 1933.

8. Kilcawley, E. J., Notes on Soil Consolidation -Rensselaer Polytechnic Institute, Troy, N. Y., June 1941.

9. Lambe, T. W., The Permeability of Fine-Grained Soils - Symposium on Permeability of Soils, A.S.T.M. Special Technical Publication No. 163, June 1954.

10. Rutledge, P. C., Recent Developments in Soil Testing Apparatus - Journal of Boston Society of Civil Engineers, Vol. XXII, No. 4, October 1935.

11. Soil Mechanics Laboratory Manual, Rensselaer Polytechnic Institute, Troy, N. Y.

12. Taylor, D. W., Fundamentals of Soil Mechanics - John Wiley & Sons, Inc., N. Y., 1948.

13. Taylor, D. W., Research on Consolidation of Clays - M.I.T. Dept. of Civil and Sanitary Engineering, Ser. 82, August 1942.

14. Terzaghi, K., Erdbaumechanik auf bodenphysikalischer Grundlage - Leipzig, Deuticke, 1925.

APPENDIX A

CONSOLIDATION

DATA

FOR TEST T-II

SOIL MECHANICS LABORATORY

RENSSELAER POLYTECHNIC INSTITUTE

TROY, N. Y.

CONSOLIDATION TEST DATA

Sample No. T-II
Date: 28 Feb. '56

Initial height of specimen = 1.488 cm.

Diameter of specimen = inside diameter of ring = 6.35 cm.

Bulk volume of specimen = 47.1 cm^3.

Wet weight of specimen (before test) = 77.15g

Dry weight of specimen = 45.1g

True specific gravity = 2.76

Wt. of pycnometer	= 82.1430g
Wt. of " and sample	= 103.8641
Wt. of " " " and water (W_2)	= 164.3221
Wt. of " " water (W_1)	= 150.4630
Wt. of dry specimen (W_o)	= 21.7211

$$G_s = \frac{W_o}{W_o + W_1 - W_2} = 2.76$$

Initial voids ratio (e_1) = 1.878

Height of soil solids in sample = .838 cm.

Initial moisture content = 63.6%

CONSOLIDATION TEST DATA

Sample No. T-II
Date 28 Feb.'56

Pressure Increment from 0 to $\frac{1}{4}$ Kg./cm.^{2}

Time Interval in Minutes	Dial Readings	
	Reading	Inches
0	0-00	.0000
.25	1-22	.0222
.50	1-27	.0227
1	1-30	.0230
2	1-33	.0233
3	1-35	.0235
5	1-37	.0237
10	1-39	.0239
15	1-40	.0240
30	1-43	.0243
60	1-46	.0246
120	1-48	.0248
240	1-50	.0250
1462	1-55	.0255

Pressure Increment from $\frac{1}{4}$ to $\frac{1}{2}$ Kg./cm.^{2}

Time Interval in Minutes	Dial Readings	
	Reading	Inches
0	1-55	.0255
.25	1-63	.0263
.50	1-58	.0258
1	1-68	.0268
2	1-70	.0270
3	1-71	.0271
5	1-73	.0273
11	1-75	.0275
15	1-76	.0276
30	1-78	.0278
60	1-80	.0280
120	1-84	.0284
149	1-84	.0284
1287	1-91	.0291

Pressure Increment from $\frac{1}{2}$ to 1 Kg./cm.^{2}

Time Interval in Minutes	Dial Readings	
	Reading	Inches
0	1-91	.0291
.25	1-119	.0319
.50	1-126	.0326
1	1-132	.0332
2	1-138	.0338
3	1-141	.0341
5	1-145	.0345
10	1-151	.0351
15	1-155	.0355
30	1-161	.0361
60	1-170	.0370
127	1-180	.0380
235	1-188	.0388
362	1-193	.0393
1590	2-08	.0408

Pressure Increment from 1 to 2 Kg./cm.^{2}

Time Interval in Minutes	Dial Readings	
	Reading	Inches
0	2-08	.0408
.25	2-65	.0465
.50	2-87	.0487
1	2-116	.0516
2	2-159	.0559
3	2-194	.0594
5	3-48	.0648
11	3-161	.0761
15	4-14	.0814
31	4-154	.0954
60	5-66	.1066
114	5-178	.1178
174	6-21	.1221
1440	6-154	.1354
1489	6-155	.1355

CONSOLIDATION TEST DATA

Sample No. T-II
Date: 28 Feb. '56

Pressure Increment from 2 to 4 Kg./cm.^{2}

Time Interval in Minutes	Dial Readings	
	Reading	Inches
0	6-155	.1355
.25	7-00	.1400
.50	7-24	.1424
1	7-52	.1452
2	7-97	.1497
3	7-130	.1530
5	7-180	.1580
10	8-56	.1656
16	8-101	.1701
30	8-146	.1746
60	8-179	.1779
120	9-01	.1801
240	9-20	.1820
1440	9-49	.1849
2755	9-53	.1853

Pressure Increment from 4 to 8 Kg./cm.^{2}

Time Interval in Minutes	Dial Readings	
	Reading	Inches
0	9-53	.1853
.25	9-109	.1909
.50	9-126	.1926
1	9-151	.1951
2	9-185	.1985
3	10-09	.2009
5	10-43	.2043
11	10-93	.2093
18	10-118	.2118
33	10-142	.2142
60	10-162	.2162
120	10-181	.2181
240	10-197	.2197
1400	11-20	.2220
2865	11-26	.2226

Pressure Increment from 8 to 16 Kg./cm.^{2}

Time Interval in Minutes	Dial Readings	
	Reading	Inches
0	11-26	.2226
.25	11-91	.2291
.50	11-112	.2312
1	11-137	.2337
2	11-170	.2370
3	11-192	.2392
5	12-17	.2417
10	12-50	.2450
15	12-74	.2474
30	12-86	.2486
60	12-100	.2500
120	12-114	.2514
1098	12-144	.2544
1524	12-149	.2549
2806	12-157	.2557

CONSOLIDATION TEST DATA

Sample No. T-II
Date: 28 Feb. '56

COEFFICIENT OF CONSOLIDATION $C_v \times 10^{-4}$ cm.^{2}/sec.

Loading Increment in Kg./cm.^{2}	Determined by Square Root of Time Fitting Method	Determined by the Logarithm of Time Fitting Method
¼		4.57
½	5.99	1.50
1	2.79	0.51
2	1.33	0.65
4	3.10	1.70
8	4.02	1.88
16	7.85	3.52

APPENDIX B

CONSOLIDATION AND PERMEABILITY DATA

FOR TEST T-III

SOIL MECHANICS LABORATORY

RENSSELAER POLYTECHNIC INSTITUTE

TROY, N. Y.

CONSOLIDATION TEST DATA

Sample No. T-III
Date: 4 Mar.'56

Initial height of specimen = 1.488 cm.

Diameter of specimen = inside diameter of ring = 6.35 cm.

Bulk volume of specimen = 47.1 cm^3.

Wet weight of specimen (before test) = 76.4g

Dry weight of specimen = 44.6g

True specific gravity = 2.70

Wt. of pycnometer	= 83.1473g
Wt. of " and sample	= 97.1762
Wt. of " " " and water (W_2)	= 158.7252
Wt. of " " water (W_1)	= 149.8876
Wt. of dry specimen (W_o)	= 14.0289

$$G_s = \frac{W_o}{W_o + W_1 - W_2} = 2.70$$

Initial voids ratio (e_1) = 1.858

Height of soil solids in sample = .917cm.

Initial moisture content = 71.4%

CONSOLIDATION TEST DATA

Sample No. T-III
Date: 4 Mar. '56

Pressure Increment from 0 to ¼ Kg./cm.^{2}

Time Interval in Minutes	Dial Readings: Reading	Dial Readings: Inches
0	0-00	.0000
.25	0-182	.0182
.50	0-186	.0186
1	0-188	.0188
2	0-190	.0190
3	0-191	.0191
5	0-192	.0192
10	0-194	.0194
15	0-195	.0195
30	0-196	.0196
60	0-198	.0198
120	1-00	.0200
240	1-02	.0202
1505	1-03	.0203

Pressure Increment from ¼ to ½ Kg./cm.^{2}

Time Interval in Minutes	Dial Readings: Reading	Dial Readings: Inches
0	1-03	.0203
.25	1-26	.0226
.50	1-29	.0229
1	1-32	.0232
2	1-35	.0235
3	1-36	.0236
5	1-38	.0238
10	1-41	.0241
15	1-43	.0243
30	1-46	.0246
65	1-50	.0250
110	1-53	.0253
1070	1-63	.0263
1640	1-64	.0264

Pressure Increment from ½ to 1 Kg./cm.^{2}

Time Interval in Minutes	Dial Readings: Readings	Dial Readings: Inches
0	1-64	.0264
.25	1-102	.0302
.50	1-109	.0309
1	1-118	.0318
2	1-125	.0325
3	1-130	.0330
5	1-137	.0337
10	1-147	.0347
17	1-155	.0355
30	1-165	.0365
60	1-178	.0378
120	1-193	.0393
240	2-11	.0411
1420	2-73	.0473
2705	2-107	.0507

Pressure Increment from 1 to 2 Kg./cm.^{2}

Time Interval in Minutes	Dial Readings: Reading	Dial Readings: Inches
0	2-107	.0507
.25	2-162	.0562
.50	2-190	.0590
1	3-31	.0631
2	3-88	.0688
3	3-133	.0733
5	4-04	.0804
10	4-109	.0909
15	4-166	.0966
30	5-51	.1051
64	5-123	.1123
123	5-176	.1176
430	6-61	.1261
1696	6-128	.1328
3163	6-158	.1358

CONSOLIDATION TEST DATA

Sample No. T-III
Date: 4 Mar. '56

Pressure Increment from 2 to 4 Kg./cm.^{2}			Pressure Increment from 4 to 8 Kg./cm.^{2}		
Time Interval in Minutes	Dial Readings		Time Interval in Minutes	Dial Readings	
	Reading	Inches		Reading	Inches
0	6-169	.1369	0	9-69	.1869
.25	7-23	.1423	.25	9-132	.1932
.50	7-49	.1449	.50	9-159	.1959
1	7-80	.1480	1	9-195	.1995
2	7-126	.1526	2	10-37	.2037
3	7-156	.1556	3	10-60	.2060
5	7-195	.1595	5	10-86	.2086
10	8-43	.1643	15	10-129	.2129
15	8-65	.1665	25	10-146	.2146
35	8-106	.1706	60	10-169	.2169
60	8-130	.1730	139	10-187	.2187
120	8-160	.1760	263	11-02	.2202
215	8-184	.1784	352	11-10	.2210
1670	9-52	.1852	1440	11-35	.2235
2771	9-69	.1869	2825	11-49	.2249

CONSOLIDATION TEST DATA

Sample No. T-III
Date: 4 Mar. '56

Mean Increment Pressure	Compression Index (C_c')	Coefficient of Compressibility (A_v')
.375 $Kg./cm.^2$	.153	.000184
.750 "	.465	.000280
1.5 "	1.440	.000434
3.0 "	.800	.000120
6.0 "	.637	.000048

COEFFICIENT OF CONSOLIDATION, $C_v \times 10^{-4}$ $cm.^2/sec.$

Loading Increment in $Kg./cm.^2$	Determined by Square Root of Time Fitting Method	Determined by the Logarithm of Time Fitting Method	Determined by Using the Direct Measurement of Permeability
¼	17.85	7.94	
½	3.38	1.67	27.7
1	7.02	.07	7.76
2	3.52	1.88	2.15
4	8.42	3.05	4.22
8	12.82	6.20	4.23

PERMEABILITY TEST DATA

Sample No. T-III
Date: 4 Mar. '56

Direct Measurements of the Coefficient of Permeability

Load Increment in Kg./cm.^{2}	Depth of Sample in cm.	e_k Void Ratio	Air Pressure in cm. of Hg.	Time Interval in Seconds	h_o in cm.	h_1 in cm.	k_d in cm./sec. x 10^{-8}	k_{20} in cm./sec. x 10^{-8}
1/4	1.437	1.755	0	1738.2	96.7	58.7	29.5	27.2
1/2	1.421	1.730	0	3395.4	103.4	56.7	17.9	16.5
"	"	"	0	2371.6	98.7	62.5	19.4	17.7
1	1.366	1.630	20.9	839.0	63.2	43.5	6.1	5.5
"	"	"	5.7	2946.8	89.4	43.1	10.45	9.53
2	1.142	1.190	0	6047.0	85.7	65.7	3.71	3.47
"	"	"	6.1	707.0	69.5	60.2	6.72	6.35
"	"	"	10.3	694.4	61.9	54.5	4.08	3.86
"	"	"	15.7	683.0	54.1	41.3	5.23	4.95
"	"	"	14.2	854.6	79.2	70.5	2.78	2.71
"	"	"	22.1	1077.0	86.3	68.4	3.27	3.14
4	1.018	.950	19.9	1837.6	80.6	62.3	1.97	1.75
"	"	"	24.2	901.6	77.7	64.8	2.38	2.14
"	"	"	26.7	1320.8	79.0	60.4	2.04	1.91
"	"	"	32.7	1010.6	68.0	49.7	2.36	2.21
"	"	"	18.7	1677.0	86.0	69.0	1.97	1.75
"	"	"	15.7	907.0	76.0	61.1	2.56	2.28
8	.916	.760	25.5	1093.4	78.7	70.9	.99	.93
"	"	"	25.6	1665.8	56.5	42.7	1.19	1.11
"	"	"	25.2	1800.4	53.4	39.9	1.14	1.08
"	"	"	24.3	1592.6	56.2	43.8	1.19	1.08
"	"	"	41.2	1428.2	54.9	36.4	1.24	1.10

APPENDIX C

CONSOLIDATION AND PERMEABILITY DATA FOR TEST T-IV

SOIL MECHANICS LABORATORY

RENSSELAER POLYTECHNIC INSTITUTE

TROY, N. Y.

CONSOLIDATION TEST DATA

Sample No. T-IV
Date: 17 Mar. '56

Initial height of specimen = 1.488 cm.

Diameter of specimen = inside diameter of ring = 6.35 cm.

Bulk volume of specimen = 47.1 cm^3.

Wet weight of specimen (before test) = 81.6g

Dry weight of specimen = 49.0g

True specific gravity = 2.73

Wt. of pycnometer	=	82.2011g
Wt. of " and sample	=	90.8022
Wt. of " " " and water (W_2)	=	155.7319
Wt. of " " water (W_1)	=	150.2786
Wt. of dry specimen (W_o)	=	8.6011

$$G_s = \frac{W_o}{W_o + W_1 - W_2} = 2.73$$

Initial voids ratio (e_1) = 1.623

Height of soil solids in sample = 1.203 cm.

Initial moisture content = 55.2%

CONSOLIDATION TEST DATA

Sample No. T-IV
Date: 17 Mar. '56

Pressure Increment from 0 to ¼ Kg./cm.^{2}

Time Interval in Minutes	Dial Readings: Reading	Dial Readings: Inches
0	0-000	.0000
.25	0-103	.0103
.50	0-115	.0115
1	0-125	.0125
2	0-134	.0134
3	0-138	.0138
5	0-142	.0142
10	0-145	.0145
15	0-146	.0146
30	0-147	.0147
60	0-148	.0148
120	0-153	.0153
1529	0-151	.0151
2700	0-150	.0150

Pressure Increment from ¼ to ½ Kg./cm.^{2}

Time Interval in Minutes	Dial Readings: Reading	Dial Readings: Inches
0	0-150	.0150
.25	0-160	.0160
.50	0-162	.0162
1	0-164	.0164
2	0-167	.0167
3	0-168	.0168
5	0-172	.0172
11	0-167	.0167
15	0-177	.0177
30	0-179	.0179
60	0-180	.0180
120	0-182	.0182
1095	0-184	.0184
1260	0-186	.0186
2820	0-191	.0191

Pressure Increment from ½ to 1 Kg./cm.^{2}

Time Interval in Minutes	Dial Readings: Reading	Dial Readings: Inches
0	0-191	.0191
.25	1-10	.0210
.50	1-15	.0215
1	1-21	.0221
2	1-29	.0229
3	1-33	.0233
5	1-38	.0238
10	1-44	.0244
15	1-47	.0247
37	1-53	.0253
60	1-56	.0256
120	1-59	.0259
271	1-62	.0262
1440	1-69	.0269

Pressure Increment from 1 to 2 Kg./cm.^{2}

Time Interval in Minutes	Dial Readings: Reading	Dial Readings: Inches
0	1-69	.0269
.25	1-99	.0299
.50	1-108	.0308
1	1-121	.0321
2	1-140	.0340
3	1-152	.0352
5	1-170	.0370
10	1-193	.0393
15	2-06	.0406
30	2-24	.0424
60	2-49	.0449
216	2-63	.0463
1190	2-92	.0492
1634	2-97	.0497
4290	2-107	.0507

CONSOLIDATION TEST DATA

Sample No. T-IV
Date: 17 Mar. '56

Pressure Increment
from 2 to 4 Kg./cm.^{2}

Time Interval in Minutes	Dial Readings	
	Reading	Inches
0	2-107	.0507
.25	2-142	.0542
.50	2-161	.0561
1	2-188	.0588
2	3-34	.0634
3	3-69	.0669
5	3-129	.0729
10	4-37	.0837
15	4-105	.0905
30	5-02	.1002
60	5-54	.1054
120	5-82	.1082
240	5-98	.1098
1440	5-123	.1123
1485*		

* Pressure ruptured specimen

CONSOLIDATION TEST DATA

Sample No. T-IV
Date: 17 Mar. '56

Mean Increment Pressure	Compression Index (C_c')	Coefficient of Compressibility (A_v')
.375 Kg./cm.^{2}	.058	.000070
.750 "	.093	.000056
1.5 "	.301	.000090
3.0 "	1.309	.000197

COEFFICIENT OF CONSOLIDATION, $C_v \times 10^{-4}$ cm.^{2}/sec.

Loading Increment in Kg./cm.^{2}	Determined by Square Root of Time Fitting Method	Determined by the Logarithm of Time Fitting Method	Determined by Using the Direct Measurement of Permeability
$\frac{1}{4}$	58.8	52.0	
$\frac{1}{2}$	28.8	11.6	15.35
1	28.9	13.2	16.22
2	9.2	4.3	7.06
4	2.1	1.8	1.65

PERMEABILITY TEST DATA

Sample No. T-IV
Date: 17 Mar. '56

Direct Measurement of Coefficient of Permeability

Load Increment in Kg./cm.^{2}	Thickness of Sample in cm.	e_k Void Ratio	Air Pressure in cm. of Hg.	Time Interval in Seconds	h_o in cm.	h_1 in cm.	k_d in cm./sec. x 10^{-8}	k_{20} in cm./sec. x 10^{-8}
$\frac{1}{4}$	1.450	1.554	0	744.6	80.9	77.3	6.05	5.18
"	1.450	1.554	13.68	900.6	52.3	45.8	2.90	2.57
"	1.450	1.554	13.14	1078.6	64.3	54.2	3.61	3.54
$\frac{1}{2}$	1.441	1.542	0	9198.2	86.3	61.6	3.42	3.32
"	1.441	1.542	15.17	1068.2	54.0	45.1	3.05	2.84
"	1.441	1.542	17.01	1250.4	54.4	40.4	3.68	3.35
"	1.441	1.542	20.47	1064.8	60.2	46.4	3.58	3.26
"	1.441	1.542	24.60	2545.4	82.8	38.5	4.05	3.68
"	1.441	1.542	18.02	1236.6	80.7	64.8	3.65	3.32
1	1.420	1.505	8.98	1058.0	80.7	75.	2.09	1.95
"	1.420	1.505	15.15	1607.8	60.0	46.7	2.90	2.72
"	1.420	1.505	20.26	1058.2	52.6	40.1	3.35	3.12
"	1.420	1.505	11.36	1949.8	65.2	50.5	3.19	2.90
"	1,420	1.505	11.70	1971.0	67.7	52.5	3.20	2.91
2	1.360	1.395	13.86	1109.2	64.8	54.5	3.18	2.96
"	1.360	1.395	17.26	1090.2	69.2	60.1	2.40	2.24
"	1.360	1.395	26.17	1076.0	68.4	54.6	2.65	2.48
"	1.360	1.395	36.17	786.2	68.5	54.6	2.91	2.72
"	1.360	1.395	42.00	537.5	70.8	59.6	3.00	2.81
4	1.203	1.122	33.04	1858.8	71.7	55.0	2.11	2.02
"	1.203	1.122	39.42	1836.0	78.0	59.8	1.26	1.20
"	1.203	1.122	46.6	2366.6	60.1	34.5	1.23	1.17
"	1.203	1.122	52.2	(rupture in sample)				

APPENDIX D

CONSOLIDATION AND PERMEABILITY DATA FOR TEST T - V

SOIL MECHANICS LABORATORY

RENSSELAER POLYTECHNIC INSTITUTE

TROY, N. Y.

CONSOLIDATION TEST DATA

Sample No. T-V
Date: 22 Mar. '56

Initial height of specimen = 1.488 cm.

Diameter of the specimen = Inside diameter of ring = 6.35 cm.

Bulk volume of specimen = 47.1 cm^2.

Wet weight of specimen (before test) = 88.6g

Dry weight of specimen = 61.4g

True specific gravity = 2.81

Wt. of pycnometer	= 82.4397g
Wt. of " and sample	= 94.5483
Wt. of " " " and water (W_2)	= 158.8578
Wt. of " " water (W_1)	= 151.0814
Wt. of dry specimen (W_o)	= 12.1086

$$G_s = \frac{W_o}{W_o + W_1 - W_2} = 2.81$$

Initial voids ratio (e_1) = 1.157

Height of soil solids in sample = 1.192 cm.

Initial moisture content = 46.2%

CONSOLIDATION TEST DATA

Sample No. T-V
Date: 22 Mar. '56

Pressure Increment from 0 to ¼ Kg./cm.^{2}

Time Interval in Minutes	Dial Readings	
	Reading	Inches
0	000	.000
.25	0-125	.0125
.50	0-139	.0139
1	0-155	.0155
2	0-177	.0177
3	0-192	.0192
5	1-16	.0216
10	1-42	.0242
15	1-50	.0250
30	1-56	.0256
90	1-63	.0263
150	1-65	.0265
240	1-67	.0267
1512	1-69	.0269
3070	1-72	.0272

Pressure Increment from ¼ to ½ Kg./cm.^{2}

Time Interval in Minutes	Dial Readings	
	Reading	Inches
0	1-72	.0272
.125	1-90	.0290
.50	1-97	.0297
1	1-105	.0305
2	1-114	.0314
3	1-120	.0320
5	1-126	.0326
10	1-131	.0331
15	1-133	.0333
30	1-137	.0337
60	1-139	.0339
120	1-141	.0341
240	1-142	.0342
1440	1-146	.0346
2765	1-148	.0348

Pressure Increment from ½ to 1 Kg./cm.^{2}

Time Interval in Minutes	Dial Readings	
	Reading	Inches
0	1-148	.0348
.25	1-177	.0377
.50	1-186	.0386
1	1-197	.0397
2	2-10	.0410
3.5	2-20	.0420
5	2-24	.0424
10	2-30	.0430
15	2-32	.0432
30	2-36	.0436
60	2-40	.0440
120	2-43	.0443
250	2-46	.0446
1535	2-50	.0450

Pressure Increment from 1 to 2 Kg./cm.^{2}

Time Interval in Minutes	Dial Readings	
	Reading	Inches
0	2-50	.0450
.25	2-88	.0488
.50	2-100	.0500
1	2-115	.0515
2	2-130	.0530
3	2-137	.0537
5	2-144	.0544
10	2-152	.0552
18	2-160	.0560
30	2-167	.0567
60	2-179	.0579
120	2-190	.0590
165	2-192	.0592
1440	3-006	.0606
2580	3-009	.0609

CONSOLIDATION TEST DATA

Sample No. T-V
Date: 22 Mar. '56

Pressure Increment from 2 to 4 Kg./cm.^{2}			Pressure Increment from 4 to 8 Kg./cm.^{2}		
Time Interval in Minutes	Dial Readings		Time Interval in Minutes	Dial Readings	
	Reading	Inches		Reading	Inches
0	3-09	.0609	0	4-50	.0850
.25	3-71	.0671	.25	4-143	.0943
.50	3-96	.0696	.50	4-177	.0977
1	3-124	.0724	1	5-18	.1018
2	3-155	.0755	2	5-58	.1058
3	3-169	.0769	3	5-75	.1075
5	3-183	.0783	5	5-90	.1090
10	3-195	.0795	10	5-103	.1103
17	4-02	.0802	15	5-109	.1109
30	4-08	.0808	30	5-118	.1118
60	4-15	.0815	60	5-125	.1125
120	4-22	.0822	120	5-132	.1132
240	4-26	.0826	240	5-141	.1141
1440	4-39	.0839	1440	5-164	.1164
2905	4-50	.0850	2685	5-164	.1164

CONSOLIDATION TEST DATA

Sample No. T-V
Date: 22 Mar. '56

Mean Increment Pressure	Compression Index (C_c')	Coefficient of Compressibility (A_v')
.375 Kg./cm.^{2}	.083	.000100
.750 "	.114	.000068
1.5 "	.147	.000044
3.0 "	.313	.000047
6.0 "	.369	.000028

COEFFICIENT OF CONSOLIDATION, $C_v \times 10^{-4}$ cm.^{2}/sec.

Loading Increment in Kg./cm.^{2}	Determined by Square Root of Time Fitting Method	Determined by the Logarithm of Time Fitting Method	Determined by Using the Direct Measurement of Permeability
$\frac{1}{4}$	9.06	9.5	
$\frac{1}{2}$	19.50	17.7	11.6
1	25.00	22.5	14.5
2	33.35	23.9	20.4
4	25.20	23.3	12.7
8	25.50	24.3	15.05

PERMEABILITY TEST DATA

Sample No. T-V
Date: 22 Mar. '56

Direct Measurement of Coefficient of Permeability

Load Increment in Kg./cm.²	Thickness of Sample in cm.	e_k Void Ratio	Air Pressure in cm. of Hg.	Time Interval in Seconds	h_c in cm.	h_1 in cm.	k_d in cm./sec. x 10^{-8}	k_{20} in cm./sec. x 10^{-8}
$\frac{1}{4}$	1.415	1.105	7.36	788.6	75.9	65.5	7.035	6.26
"	"	"	5.25	4228.2	68.1	30.5	6.67	5.87
"	"	"	0	3577.0	96.2	67.4	9.14	8.14
$\frac{1}{2}$	1.400	1.030	0	2601.8	88.1	72.3	6.88	6.43
"	"	"	11.23	2203.5	77.5	45.0	6.22	5.87
"	"	"	11.50	1796.8	78.2	51.9	5.92	5.68
"	"	"	9.90	1401.4	77.7	60.0	5.53	5.31
"	"	"	16.05	1159.2	54.0	34.7	5.63	5.63
1	1.374	.947	0	5374.0	77.1	59.0	4.46	4.33
"	"	"	11.75	1802.6	61.4	40.5	4.81	4.47
"	"	"	13.60	1773.4	85.0	58.1	5.16	4.70
"	"	"	18.75	1615.6	62.9	32.5	5.40	4.85
"	"	"	18.70	1238.0	62.0	37.5	5.71	5.14
2	1.334	.932	13.35	1843.0	67.3	45.1	4.28	3.81
"	"	"	16.20	1892.0	76.7	48.5	4.44	3.95
"	"	"	20.05	1797.0	78.6	45.4	4.68	4.25
"	"	"	24.00	1438.2	77.0	45.0	4.82	4.28
"	"	"	24.70	1686.0	82.4	42.0	5.16	4.59
4	1.272	.843	12.65	1846.0	79.9	64.2	2.82	2.76
"	"	"	17.00	2083.0	82.0	58.3	3.04	2.95
"	"	"	24.60	1811.8	72.5	44.0	3.25	3.13
"	"	"	29.60	2149.6	74.1	34.0	3.32	3.19
"	"	"	35.60	1783.0	70.9	31.4	3.38	3.24

Load Increment in Kg./cm.^{2}	Thickness of Sample in cm.	e_k Void Ratio	Air Pressure in cm.of Hg.	Time Interval in Seconds	h_c in cm.	h_1 in cm.	k_d in cm./sec. x 10^{-8}	k_{20} in cm./sec. x 10^{-8}
8	1.1925	.763	18.00	2089.5	63.9	43.2	2.70	2.42
"	"	"	20.55	4060.5	80.0	37.1	2.36	2.17
"	"	"	27.65	3603.0	74.5	28.0	2.29	2.11
"	"	"	32.90	1903.2	62.0	33.0	2.36	2.16
"	"	"	38.45	1794.0	75.6	43.4	2.33	2.18
"	"	"	42.80	1856.0	72.0	35.6	2.33	2.18
"	"	"	48.90	1858.4	75.5	34.4	2.32	2.17
"	"	"	54.05	1816.5	76.0	31.5	2.38	2.25
"	"	"	54.00	1832.5	76.8	33.0	2.28	2.16

APPENDIX E

CONSOLIDATION AND PERMEABILITY DATA FOR TEST T-VI

SOIL MECHANICS LABORATORY

RENSSELAER POLYTECHNIC INSTITUTE

TROY, N. Y.

CONSOLIDATION TEST DATA

Sample No. T-VI
Date: 3 Apr.'56

Initial height of specimen = 1.488 cm.

Diameter of the specimen = inside diameter of ring = 6.35 cm.

Bulk volume of specimen = 47.1 cm^3.

Wet weight of specimen = 89.0g

Dry weight of specimen = 64.8g

True specific gravity = 2.72

Wt. of pycnometer	=	82.1650g
Wt. of " and sample	=	98.7368
Wt. of " " " and water (W_2)	=	160.7175
Wt. of " " water (W_1)	=	150.2493
Wt. of dry specimen (W_o)	=	16.5718

$$G_s = \frac{W_o}{W_o + W_1 - W_2} = 2.72$$

Initial voids ratio (e_1) = .975

Height of soil solids in sample = 1.142 cm.

Initial moisture content = 48.4%

CONSOLIDATION TEST DATA

Sample No. T-VI
Date: 3 Apr. '56

Pressure Increment from 0 to $\frac{1}{4}$ Kg./cm.^{2}

Time Interval in Minutes	Dial Readings	
	Reading	Inches
0	000	.0000
.25	0-126	.0126
.50	0-138	.0138
1	0-153	.0153
2	0-171	.0171
3	0-184	.0184
5	1-02	.0202
10	1-17	.0217
15	1-23	.0223
30	1-27	.0227
60	1-29	.0229
130	1-31	.0231
240	1-32	.0232
1205	1-35	.0235

Pressure Increment from $\frac{1}{4}$ to $\frac{1}{2}$ Kg./cm.^{2}

Time Interval in Minutes	Dial Readings	
	Reading	Inches
0	1-35	.0235
.25	1-51	.0251
.50	1-55	.0255
1	1-62	.0262
2	1-69	.0269
3	1-74	.0274
5	1-78	.0278
10	1-83	.0283
15	1-85	.0285
30	1-88	.0288
60	1-91	.0291
120	1-94	.0294
240	1-96	.0296
1440	1-101	.0301
2880	1-108	.0305

Pressure Increment from $\frac{1}{2}$ to 1 Kg./cm.^{2}

Time Interval in Minutes	Dial Readings	
	Reading	Inches
0	1-105	.0305
.25	1-128	.0328
.50	1-135	.0335
1	1-143	.0343
2	1-152	.0352
3	1-157	.0357
5	1-162	.0362
10	1-167	.0367
15	1-169	.0369
30	1-174	.0374
60	1-177	.0377
120	1-180	.0380
240	1-184	.0384
1440	1-193	.0393
2880	1-196	.0396

Pressure Increment from 1 to 2 Kg./cm.^{2}

Time Interval in Minutes	Dial Readings	
	Reading	Inches
0	1-196	.0396
.25	2-28	.0428
.50	2-36	.0436
1	2-47	.0447
2	2-60	.0460
3	2-66	.0466
5	2-73	.0473
10	2-80	.0480
15	2-84	.0484
30	2-90	.0490
60	2-95	.0495
120	2-103	.0503
240	2-106	.0506
1470	2-118	.0518
2820	2-123	.0523

CONSOLIDATION TEST DATA

Sample No. T-VI
Date: 3 Apr. '56

Time Interval in Minutes	Dial Readings (Pressure Increment from 2 to 4 Kg./cm.^{2})		Time Interval in Minutes	Dial Readings (Pressure Increment from 4 to 8 Kg./cm.^{2})	
	Reading	Inches		Reading	Inches
0	2-123	.0523	0	3-117	.0717
.25	2-170	.0570	.25	3-181	.0781
.50	2-185	.0585	.50	4-04	.0804
1	3-05	.0605	1	4-30	.0830
2	3-25	.0625	2	4-57	.0857
3	3-34	.0634	3	4-70	.0870
5	3-45	.0645	5	4-83	.0883
10	3-56	.0656	10	4-100	.0900
15	3-62	.0662	15	4-109	.0909
30	3-71	.0671	30	4-120	.0920
60	3-79	.0679	60	4-130	.0930
120	3-87	.0687	115	4-139	.0937
232	3-93	.0693	250	4-148	.0948
1275	3-113	.0713	1440	4-168	.0968
2845	3-117	.0717	2880	4-175	.0975

CONSOLIDATION TEST DATA

Sample No. T-VI
Date: 3 Apr. '56

Mean Increment Pressure	Compression Index (C_c')	Coefficient of Compressibility (A_v')
.375 Kg./cm.^{2}	.061	.0000735
.750 "	.094	.0000566
1.5 "	.127	.0000382
3.0 "	.188	.0000283
6.0 "	.285	.0000215

COEFFICIENT OF CONSOLIDATION, $C_v \times 10^{-4}$ cm.^{2}/sec.

Loading Increment in Kg./cm.^{2}	Determined by Square Root of Time Fitting Method	Determined by the Logarithm of Time Fitting Method	Determined by Using the Direct Measurement of Permeability
¼	18.45	13.21	
½	22.50	19.75	14.55
1	27.10	23.35	16.18
2	21.28	18.92	20.13
4	26.90	26.50	20.43
8	34.50	23.70	30.85

PERMEABILITY TEST DATA

Sample No. T-VI
Date: 3 Apr.'56

Direct Measurement of Coefficient of Permeability

Load Increment in Kg./cm.²	Thickness of Sample in cm.	e_k Void Ratio	Air Pressure in cm. of Hg.	Time Interval in Seconds	h_o in cm.	h_1 in cm.	k_d in cm./sec. x 10^{-8}	k_{20} in cm./sec. x 10^{-8}
$\frac{1}{4}$	1.4288	.895	0	7423.4	86.6	57.5	5.12	5.02
"	"	"	4.6	2469.2	71.1	55.9	4.46	4.42
$\frac{1}{2}$	1.4115	.874	0	1905.0	81.8	65.5	10.82	9.85
"	"	"	5.3	3067.7	71.6	50.1	4.66	4.24
"	"	"	8.55	3103.0	65.0	35.0	5.26	4.74
"	"	"	8.5	2295.3	79.3	54.8	5.33	4.80
1	1.3882	.841	8.65	4187.8	82.0	42.1	4.70	4.09
"	"	"	14.05	1984.	81.0	52.8	4.92	4.35
"	"	"	19.1	1917.8	71.0	37.1	4.98	4.44
"	"	"	22.0	1818.2	70.8	35.0	4.96	4.42
"	"	"	21.93	1857.8	76.7	40.0	4.88	4.35
2	1.356	.799	14.73	2343.3	58.7	32.0	3.98	3.66
"	"	"	20.4	1905.6	65.4	35.4	4.16	3.78
"	"	"	20.3	2160.4	80.6	45.5	4.16	3.78
"	"	"	25.8	1805.0	67.5	31.5	4.26	3.88
"	"	"	29.2	1806.7	73.8	33.3	4.30	3.91
4	1.307	.736	20.85	1808.2	78.6	53.6	3.29	2.99
"	"	"	23.75	1717.5	79.4	53.1	3.25	2.96
"	"	"	0	10580.0	80.9	55.7	3.00	2.81
"	1.3057	.732	23.95	1905.0	68.8	41.5	3.09	2.87
"	"	"	14.0	3974.4	86.1	48.0	3.10	2.88
8	1.242	.648	33.18	2280.0	72.3	37.4	2.39	2.18
"	"	"	33.03	2119.3	81.4	49.3	2.33	2.10
"	"	"	33.05	2601.0	82.5	42.7	2.34	2.11
"	"	"	33.05	2445.4	83.2	46.0	2.38	2.15
"	"	"	40.3	1803.6	70.9	38.4	2.36	2.15

APPENDIX F

CONSOLIDATION AND PERMEABILITY DATA FOR TEST T-VII

SOIL MECHANICS LABORATORY

RENSSELAER POLYTECHNIC INSTITUTE

TROY, N. Y.

CONSOLIDATION TEST DATA

Sample No. T-VII
Date: 4 Apr. '56

Initial height of specimen = 1.488 cm.

Diameter of the specimen = inside diameter of the ring = 6.35 cm.

Bulk volume of specimen = 47.1 cm^3.

Wet weight of specimen = 88.8g

Dry weight of specimen = 63.8g

True specific gravity = 2.74

Wt. of pycnometer	= 82.4392g
Wt. of " and sample	= 99.1345
Wt. of " " " and water (W_2)	= 161.5772
Wt. of " and water (W_1)	= 150.9768
Wt. of dry specimen (W_o)	= 16.6953

$$G_s = \frac{W_o}{W_o + W_1 - W_2} = 2.74$$

Initial voids ratio (e_1) = 1.022

Height of soil solids in sample = 1.142 cm.

Initial moisture content = 40.3%

CONSOLIDATION TEST DATA

Sample No. T-VII
Date: 4 Apr. '56

Pressure Increment from 0 to ¼ Kg./cm.^{2}

Time Interval in Minutes	Dial Readings	
	Reading	Inches
0	0-000	.0000
.25	1-07	.0207
.50	1-13	.0213
1	1-21	.0221
2	1-31	.0231
3	1-38	.0238
5	1-47	.0247
10	1-56	.0256
15	1-59	.0259
30	1-63	.0263
60	1-166	.0266
120	1-67	.0267
240	1-168	.0268
1440	1-71	.0271

Pressure Increment from ¼ to ½ Kg./cm.^{2}

Time Interval in Minutes	Dial Readings	
	Reading	Inches
0	1-71	.0271
.25	1-87	.0287
.50	1-91	.0291
1	1-97	.0297
2	1-104	.0304
3	1-108	.0308
5	1-114	.0314
10	1-120	.0320
15	1-122	.0322
30	1-125	.0325
60	1-128	.0328
120	1-130	.0330
240	1-132	.0332
1440	1-137	.0337

Pressure Increment from ½ to 1 Kg./cm.^{2}

Time Interval in Minutes	Dial Readings	
	Reading	Inches
0	1-149	.0349
.25	1-167	.0367
.50	1-172	.0372
1	1-180	.0380
2	1-189	.0389
3	1-194	.0394
5	2-000	.0400
10	2-05	.0405
15	2-07	.0407
31	2-11	.0411
60	2-14	.0414
120	2-16	.0416
240	2-20	.0420
3000	2-26	.0426

Pressure Increment from 1 to 2 Kg./cm.^{2}

Time Interval in Minutes	Dial Readings	
	Reading	Inches
0	2-27	.0427
.25	2-56	.0456
.50	2-65	.0465
1	2-76	.0476
2	2-90	.0490
3	2-97	.0497
5	2-105	.0505
10	2-113	.0513
15	2-118	.0518
30	2-124	.0524
60	2-130	.0530
120	2-136	.0536
240	2-142	.0542
1560	2-155	.0555
2825	2-160	.0560

CONSOLIDATION TEST DATA

Sample No. T-VII
Date: 4 Apr. '56

Pressure Increment from 2 to 4 Kg./cm.^{2}

Time Interval in Minutes	Dial Readings	
	Reading	Inches
0	2-160	.0560
.25	3-10	.0610
.50	3-24	.0624
1	3-42	.0642
2	3-62	.0662
3	3-74	.0674
5	3-88	.0688
10	3-101	.0701
15	3-107	.0707
30	3-116	.0716
60	3-125	.0725
120	3-133	.0733
280	3-144	.0744
1490	3-157	.0757
1722	3-160	.0760
3080	3-166	.0766

Pressure Increment from 4 to 8 Kg./cm.^{2}

Time Interval in Minutes	Dial Readings	
	Reading	Inches
0	3-166	.0766
.25	4-31	.0831
.50	4-54	.0854
1	4-82	.0882
2	4-115	.0915
3	4-133	.0933
5	4-155	.0955
10	4-173	.0973
15	4-182	.0982
30	4-196	.0996
90	5-15	.1015
120	5-20	.1020
240	5-29	.1029
1455	5-48	.1048
2865	5-58	.1058

Pressure Increment from 8 to 16 Kg./cm.^{2}

Time Interval in Minutes	Dial Readings	
	Reading	Inches
0	5-58	.1058
.25	5-143	.1143
.50	5-169	.1169
1	6-03	.1203
2	6-39	.1239
3	6-58	.1258
7	6-85	.1285
10	6-94	.1294
15	6-102	.1302
30	6-114	.1314
60	6-124	.1324
155	6-137	.1337
240	6-143	.1343
1250	6-160	.1360
1440	6-161	.1361

CONSOLIDATION TEST DATA

Sample No. T-VII
Date: 4 Apr. '56

Mean Increment Pressure	Compression Index (C_c')	Coefficient of Compressibility (A_v')
.375 Kg./cm.^{2}	.069	.0000830
.750 "	.092	.0000554
1.5 "	.124	.0000373
3.0 "	.220	.0000331
6.0 "	.323	.0000243
12.0 "	.344	.0000129

COEFFICIENT OF CONSOLIDATION, $C_v \times 10^{-4}$ cm.^{2}/sec.

Loading Increment in Kg./cm.^{2}	Determined by Square Root of Time Fitting Method	Determined by The Logarithm of Time Fitting Method	Determined by Using the Direct Measurement of Permeability
$\frac{1}{4}$	19.05	10.05	
$\frac{1}{2}$	18.56	12.80	8.56
1	20.67	13.82	11.65
2	25.82	21.33	14.48
4	23.42	16.98	11.98
8	18.75	15.83	9.83
16	22.30	18.39	10.62

PERMEABILITY TEST DATA

Sample No. T-VII
Date: 4 Apr. '56

Direct Measurement of Coefficient of Permeability *

Load Increment in Kg./cm.²	Thickness of Sample in cm.	e_k Void Ratio	Air Pressure in cm. of Hg.	Time Interval in Seconds	h_o in cm.	h_1 in cm.	k_d in cm./sec. x 10^{-8}	k_{20} in cm./sec. x 10^{-8}
1/4	1.420	.930	4.55	2474.3	79.0	66.5	3.55	3.36
"	"	"	5.9	4577.7	75.0	48.8	3.67	3.43
"	"	"	6.9	3374.0	79.2	56.2	3.82	3.51
1/2	1.402	.906	6.33	3104.6	68.4	49.4	3.78	3.37
"	"	"	8.9	2317.6	71.2	53.2	3.77	3.36
"	"	"	8.5	1641.8	70.3	58.5	3.60	3.20
1	1.380	.875	9.4	4190.6	69.8	41.2	3.28	2.98
"	"	"	14.48	2312.5	77.5	55.0	3.56	3.26
"	"	"	19.95	1864.8	80.9	55.2	3.55	3.25
"	"	"	22.3	1842.0	80.9	55.2	3.31	3.03
"	"	"	22.2	1901.5	82.3	55.9	3.27	3.06
2	1.347	.830	15.07	2388.5	67.0	46.0	2.90	2.58
"	"	"	20.4	2383.0	76.1	48.0	2.96	2.66
"	"	"	20.4	2225.8	84.8	58.5	2.91	2.62
"	"	"	25.75	2229.2	84.1	52.0	2.95	2.68
"	"	"	29.5	929.0	79.3	63.9	3.00	2.73
4	1.295	.761	21.55	1909.3	63.8	44.7	2.44	2.21
"	"	"	23.9	4943.8	83.9	32.2	2.21	1.99
"	"	"	26.6	4052.8	82.0	35.5	2.25	2.03
"	1.294	.759	21.0	3600.8	86.7	52.5	2.22	2.02
"	"	"	21.95	4433.4	84.4	41.0	2.30	2.09
8	1.222	.661	33.03	3552.7	85.0	51.0	1.43	1.33
"	"	"	33.1	3640.5	86.2	51.3	1.44	1.33
"	"	"	33.1	3638.0	86.0	51.4	1.42	1.32
"	"	"	33.0	3730.0	81.5	46.3	1.44	1.35
16	1.142	.552	50.15	2671.1	76.0	52.1	.875	.840
"	"	"	50.17	4039.3	83.0	47.3	.852	.815
"	"	"	50.13	5258.1	84.8	38.9	.837	.786

* All tests were conducted using air pressure to provide the head

APPENDIX G

CONSOLIDATION AND PERMEABILITY DATA FOR TEST T-VIII

SOIL MECHANICS LABORATORY

RENSSELAER POLYTECHNIC INSTITUTE

TROY, N. Y.

CONSOLIDATION TEST DATA

Sample No. T-VIII
Date: 16 Apr. '56

Initial height of specimen = 1.468 cm.

Diameter of the specimen = inside diameter of ring = 6.35 cm.

Bulk volume of specimen = 47.1 cm^3.

Wet weight of specimen (before test) = 78.3g

Dry weight of specimen = 47.3g

True specific gravity (G_s) = 2.72

Wt. of pycnometer	= 82.1437g
Wt. of " and sample	= 102.8575
Wt. of " " " and water (W_2)	= 163.3420
Wt. of " " water (W_1)	= 150.2613
Wt. of dry specimen (W_o)	= 20.7138

$$G_s = \frac{W_o}{W_o + W_1 - W_2} = 2.72$$

Initial voids ratio (e_1) = 1.715

Height of soil solids in sample = .915 cm.

Initial moisture content = 52.3%

CONSOLIDATION TEST DATA

Sample No. T-VIII
Date: 16 Apr. '56

Pressure Increment from 0 to ¼ Kg./cm.^{2}

Time Interval in Minutes	Dial Readings	
	Reading	Inches
0	0-000	.0000
.25	0-104	.0104
.50	0-113	.0113
1	0-125	.0125
2	0-138	.0138
3	0-145	.0145
5	0-153	.0153
10	0-160	.0160
15	0-162	.0162
30	0-166	.0166
60	0-168	.0168
125	0-125	.0125
1070	0-181	.0181
1440	0-182	.0182
2690	0-186	.0186

Pressure Increment from ¼ to ½ Kg./cm.^{2}

Time Interval in Minutes	Dial Readings	
	Reading	Inches
0	0-186	.0186
.25	1-03	.0203
.50	1-10	.0210
1	1-17	.0217
2	1-28	.0228
4	1-41	.0241
5	1-45	.0245
10	1-57	.0257
15	1-63	.0263
45	1-78	.0278
60	1-82	.0282
120	1-89	.0289
240	1-96	.0296
330	1-99	.0299
1260	1-110	.0310

Pressure Increment from ½ to 1 Kg./cm.^{2}

Time Interval in Minutes	Dial Readings	
	Reading	Inches
0	1-110	.0310
.25	1-143	.0343
.50	1-156	.0356
1	1-172	.0372
2	1-197	.0397
3	2-15	.0415
5	2-38	.0438
10	2-71	.0471
15	2-88	.0488
30	2-113	.0513
60	2-133	.0533
120	2-150	.0550
240	2-163	.0563
1440	2-189	.0589
1800	2-191	.0591

Pressure Increment from 1 to 2 Kg./cm.^{2}

Time Interval in Minutes	Dial Readings	
	Reading	Inches
0	2-191	.0591
.25	3-33	.0633
.50	3-57	.0657
1	3-80	.0680
2	3-118	.0718
3	3-148	.0748
5	3-190	.0790
10	4-54	.0854
15	4-87	.0887
30	4-131	.0931
77	4-173	.0973
125	4-191	.0991
195	5-06	.1006
1330	5-49	.1049
2550	5-62	.1062

CONSOLIDATION TEST DATA

Sample No. T-VIII
Date: 16 Apr. '56

Pressure Increment from 2 to 4 Kg.cm.^{2}

Time Interval in Min-	Dial Readings	
	Reading	Inches
0	5-62	.1062
.25	5-116	.1116
.50	5-141	.1141
1	5-174	.1174
2	6-23	.1223
3	6-59	.1259
5	6-108	.1308
13	6-188	.1388
15	6-198	.1398
30	7-31	.1431
60	7-55	.1455
120	7-73	.1473
195	7-83	.1483
1380	7-117	.1517
1670	7-118	.1518
2675	7-121	.1512

Pressure Increment from 4 to 8 Kg./cm.^{2}

Time Interval in Min-	Dial Readings	
	Reading	Inches
0	7-121	.1521
.25	7-180	.1580
.50	8-03	.1603
1	8-35	.1635
2	8-77	.1677
3	8-107	.1707
5	8-147	.1747
10	8-193	.1793
15	9-13	.1813
30	9-36	.1836
60	9-55	.1855
120	9-71	.1871
240	9-85	.1885
455	9-96	.1896
1615	9-111	.1911

Pressure Increment from 8 to 16 Kg./cm.^{2}

Time Interval in Minutes	Dial Readings	
	Reading	Inches
0	9-111	.1911
.25	9-171	.1971
.50	9-193	.1993
1	10-24	.2024
2	10-61	.2061
3	10-85	.2085
6	10-123	.2123
10	10-145	.2145
15	10-158	.2158
30	10-177	.2177
60	10-194	.2194
120	11-10	.2210
240	11-24	.2224
495	11-35	.2235
1680	11-51	.2251
2925	11-55	.2255

CONSOLIDATION TEST DATA

Sample No. T-VIII
Date: 16 Apr. '56

Mean Increment Pressure	Compression Index (C_c')	Coefficient of Compressibility (A_v')
.375 Kg./cm.^{2}	.208	.000250
.750 "	.447	.000269
1.5 "	.773	.000232
3.0 "	.710	.000107
6.0 "	.594	.000045
12.0 "	.520	.000019

COEFFICIENT OF CONSOLIDATION, $C_v \times 10^{-4}$ cm.^{2}/sec.

Loading Increment in Kg./cm.^{2}	Determined by Square Root of Time Fitting Method	Determined by the Logarithm of Time Fitting Method	Determined by Using the Direct Measurement of Permeability
$\frac{1}{4}$	27.48	20.03	
$\frac{1}{2}$	15.81	7.28	9.04
1	8.56	3.76	5.18
2	4.77	3.08	3.40
4	5.54	3.87	3.54
8	6.69	3.95	4.17
16	7.76	4.26	4.20

PERMEABILITY TEST DATA

Sample No. T-VIII
Date: 16 Apr. '56

Direct Measurement of Coefficient of Permeability *

Load Increment in Kg./cm.²	Thickness of Sample in cm.	e_k Void Ratio	Air Pressure in cm. of Hg.	Time Interval in Seconds	h_o in cm.	h_1 in cm.	k_d in cm./sec. x 10^{-8}	k_{20} in cm./sec. x 10^{-8}
$\frac{1}{4}$	1.442	1.632	5.9	2753.2	75.0	35.5	9.76	9.13
"	"	"	6.2	1807.6	74.0	44.4	10.55	9.86
"	"	"	6.2	1153.8	77.3	56.9	10.72	10.03
$\frac{1}{2}$	1.409	1.573	11.3	2059.6	68.7	28.8	8.62	8.19
"	"	"	11.1	2248.2	75.4	31.5	8.60	8.00
"	"	"	11.0	1981.0	74.0	35.5	8.56	8.00
"	"	"	11.0	2302.9	78.5	32.5	8.80	8.22
"	"	"	11.2	2228.5	79.5	34.7	8.90	8.31
1	1.338	1.444	20.2	1613.8	75.0	39.0	5.68	5.39
"	"	"	20.2	1797.1	80.0	39.0	5.80	5.51
"	"	"	20.2	1475.0	78.5	44.5	5.81	5.52
"	"	"	0	4801.5	67.5	50.3	5.34	5.00
2	1.222	1.232	25.5	2199.5	73.5	33.0	3.62	3.19
"	"	"	25.4	1816.3	74.9	41.5	3.58	3.22
"	"	"	25.6	2443.2	83.0	38.5	3.45	3.17
"	"	"	25.6	1312.6	80.0	56.0	3.48	3.17
4	1.103	1.015	35.2	2747.4	79.5	39.5	1.91	1.72
"	"	"	35.4	2743.5	80.5	40.8	1.88	1.69
"	"	"	35.4	2859.4	78.3	37.0	1.90	1.72
"	"	"	35.4	3538.5	82.0	32.0	1.86	1.69
8	1.002	.832	50.3	2834.6	77.7	45.0	1.05	.98
"	"	"	50.1	2937.8	82.0	47.0	1.02	.95
"	"	"	50.6	2586.8	82.0	50.9	1.01	.94
16	.9165	.675	69.1	3774.8	81.4	46.2	.49	.46

* Air Pressures Held Fairly constant for tests during each loading increment.

Thesis 32845
P39 Petersen
An analysis of the consolidation - permeability characteristics of clay soils.